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THE AMERICAN ARCHITECT AND BUILDING NEWS

VOL. XXIII. REGULAR EDITION NO. 641.

ARCHITECTURE

ENGINEERING

DECORATION

CONSTRUCTION

TICKNOR & COMPANY

BOSTON MASS.

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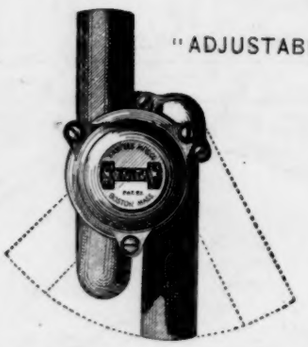
(Gelatine Print issued only with Imperial Edition.)
U. S. Court-House and Post-Office, San Antonio, Tex.
Competitive Design for the City Hall, Minneapolis, Minn.
Some Old Bells.

THE SANITAS TRAP.



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THE AMERICAN ARCHITECT AND BUILDING NEWS.

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THERE is something interesting in the way in which Americans take up the idea of sharing the profits of manufacturing among the operatives in the factory, which has now attained a wide-spread popularity here. American manufacturers are almost always men who have themselves been operatives, and they reason with their employés in an unassuming, sensible way which is often very instructive. Among others, the Springfield Foundry Company of Massachusetts has just paid a dividend of two and one-half per cent as an addition to the wages earned by each man who has been with the Company during the year. Although this amounts to a present of twenty dollars or so to each person employed, at a season when such extra income is particularly acceptable, the directors of the Company, in their circular announcing the dividend, rather apologize for its smallness, and, while acknowledging the disposition shown by most of the men to do their part in earning it, express the hope that another year's business will show a better result, and point out the most effective way to accomplish this. In their judgment the most serious drain upon the surplus profits which might be used for paying dividends comes from the carelessness of moulders and others, who needlessly turn out defective castings. In every foundry there is a certain amount of loss from this cause, but we imagine that the Springfield Company's men will be rather surprised to learn that the cost of making good their own careless work amounted during the past year to twice as much as the dividend; and that if they had all done their work as well as they could, their share of profits, without any extra hours of labor, or any extra exertion beyond a little care, would have averaged sixty dollars in place of twenty. This difference will seem to the dullest workman well worth saving another year, and the Company's circular reinforces the moral by pointing out that the loss on an imperfect casting is sometimes as great as the profit on a good one; so that if a man loses a casting which takes one hour to mould, he must work his best all the rest of the day barely to cover the loss from his careless hour, without earning anything as profit for that day, either for himself or his employers. This, it seems to us, is very well put, presenting the lesson that profit in business depends on constant care and economy in the smallest details in a forcible and simple way. The circular closes by reminding the men that they have a personal interest in the business; that every minute of time and every cent's worth of property saved increases their profits; and that every pound of poor and rough casting injures the reputation and business of the foundry, while every pound of good casting helps both, and increases the present and prospective profits of the workmen.

"THERMUS," the clever expert correspondent of the *Engineering and Building Record*, describes in a recent number a hot-water heating apparatus which he has applied to a small city house with very satisfactory results. The house in question was originally provided with one of those feeble contrivances, the "Baltimore heater," which, as readers in places where house-warming is scientifically carried on may need to know, consists of a sort of flat stove set in an ordinary fireplace and having a smoke-pipe led up the flue, around which a certain amount of partially-warmed air struggles up into the rooms above. Finding it inconvenient to be obliged to wear his overcoat at breakfast in cold weather, to say nothing of the objections to leaving his family to shiver all day in a half-warmed house, "Thermus" bought a No. 22 Hitchings base-burning greenhouse boiler and set it in the corner of his dining-room, which, as is common in New York houses, is the front basement room. From the connections provided on the boiler, he then carried a line of flow and return pipes, supplying four Bundy hot-water radiators, two of which were placed in the entrance hall, while the other two were set near the windows in the front and rear first-story rooms. A small expansion-tank was connected with the upper end of the loop of pipe and the apparatus was then complete, and it has continued to warm the house comfortably in the coldest weather, with a consumption of less than fifty pounds of coal per day, and a maximum temperature of one hundred and fifty degrees in the flow-pipe, as measured by the thermometer inserted for the purpose. Owing, no doubt, to the liberal provision of heating-surface in the first-story hall, the chambers in the second story are sufficiently warm without any radiators specially devoted to them, and as the apparatus runs day and night, the house does not get chilled. As compared with the old heater, which, with a consumption of coal more than fifty per cent greater, failed to warm even the room in which it stood, this simple hot-water system seems to have been very successful, and while commending "Thermus's" letter, which is to be found in the issue of the *Engineering and Building Record* for January 7th, to the attention of persons interested in the subject, we feel ourselves moved to say a little on our own account in regard to what is certainly fast becoming the most popular mode of house-heating.

HAVING ourselves used a No. 22 Hitchings base-burner boiler for several years, we can confirm all that "Thermus" says of its virtues. Small as it is, the whole affair being only twenty-one inches in diameter, and forty-two inches high, fire can be kept in it continuously for almost any length of time. With our own boiler, which, however, has a comparatively small duty to perform, the consumption of coal does not average more than twenty-five pounds a day, and, in all but the coldest weather, one supply of coal in twenty-four hours, with a corresponding shaking out and removal of ashes, is sufficient to keep an equable heat all through the hot-water system day and night; and with good coal we have kept fire continuously from October to April. The freedom of the fire in this, as in other hot-water boilers, from the variations and uncertainties to which small hard-coal fires are ordinarily subject, is probably due to the conservative influence of the large body of water circulating around the fire-pot, which tends powerfully to maintain the coal at an equal temperature, and to protect it from the sudden chills which would extinguish a similar fire in an ordinary stove. Whether the excellence of the results to be obtained with the boilers is, however, all that is necessary to prove the superiority of hot-water heating, is doubtful. In the house which "Thermus" describes, the cost of the apparatus for warming three rooms and the hall, is set down as one hundred and seventy-five dollars, as a minimum, and as all the heating is by direct radiation, no fresh-air is admitted unless the windows are opened. Now, according to the plan given in "Thermus's" letter, there would be no difficulty in setting in the basement a small furnace, which, at no greater first cost, would warm all the rooms now warmed by the hot-water, and probably the second story bath-room in addition, and, besides heat, would supply fresh-air. Whether the expense of operating the furnace would be as small as that of the hot-water boiler is very doubtful, but fresh-air is a luxury, to be paid for like other luxuries, and a good furnace will supply it as cheaply as any apparatus in common use. Moreover, there is an unpleasant possibility that the hot-water pipes, if the house should be left to itself for a day or two, may freeze, and either they

or the boiler will burst in consequence, the result in either case being a long bill for repairs, and a house uninhabitable for a time. Having ourselves had this experience, we can speak with confidence upon the point, and on this, as well as other grounds, we may say that, for small houses, several improvements in the application of hot-water heating are needed, before the system can compete successfully with that which employs a hot-air furnace. One of the most important improvements yet to be made should lie in the reduction of the cost of radiators. In "Thermus's" house the four radiators are valued at fifty-six dollars, or forty cents per square foot of surface. This, although moderate in comparison with the prices once charged, is, in our opinion, about twice what it should be. Cast-iron pipe, such as is used for greenhouse heating, pays a large profit at twenty cents per square foot of radiating-surface, and there seems to be no reason why buckled plates, or some simple form of cast-iron radiator, should not be made at as low a rate. Moreover, to compete successfully with furnace-heating, hot-water radiators for dwellings ought to be made to take in a fresh-air supply from out-of-doors, on the direct-indirect plan, and deliver it warmed into the room. There ought to be no serious difficulty in designing simple radiators of this sort, which would do more to solve the problem of convenient house-heating, than anything yet devised. The next thing would be to compound some solution which could be used instead of pure water in the heating system, and which would not freeze so as to burst the pipes or boiler under any circumstances. The well-known hot-water car-heaters use strong brine instead of water. This has the advantage of obviating danger from frost, as well as the incidental one of raising slightly the maximum temperature of the radiating surfaces; but it has the great disadvantage of corroding the pipes, and the house-building world still awaits the mixture which will offer the advantages of brine, without its defects. When this comes, together with the inexpensive radiator which will take in a six-inch stream of air at zero from the outside of the house, and deliver it at ninety degrees into the room, and Mr. Fletcher's boiler, which, by the aid of copper pegs driven through the bottom, presents an efficiency several times as great as that of the old kind, we may expect to see hot-water apparatus supersede furnaces in small houses, as it already has to a considerable extent in large ones.

ANOTHER very fatal theatre fire occurred a few days ago in Portugal, where the Banquet Theatre at Oporto took fire on the stage, and was destroyed, about a hundred persons being either suffocated by the smoke and gas, or crushed in the struggle to escape. The cause of the fire is thought to have been a leakage of gas from a pipe under the stage, by which enough gas was accumulated to form an inflammable mixture with the air; and in the midst of the performance a violent explosion took place, extinguishing most of the lights in the theatre, and scattering fire in all directions. As usual, the persons nearest the doors escaped, but those behind them, in their eagerness to get out, barricaded the exits, so that they were crushed, while they prevented others from escaping. The occupants of the upper tiers of boxes, seeing the impossibility of reaching the doors, jumped from the windows, saving their lives at the cost of bruises and broken limbs. The stage exits seem to have been in unusually good condition, and the performers were mostly saved. If the year should go on as it has begun, the season of 1887–88 will long be famous for theatre fatalities. For several years the disasters at Nice and Vienna were the only very notable ones occurring in Europe, but since the present theatrical season began we have had three very serious and fatal conflagrations, at Exeter, Paris and Oporto, with some months remaining for further catastrophes.

ANOTHER sad accident occurred in New York recently, where a handsome apartment-house on one of the best streets in the city and reputed to be nearly fireproof took fire in the third story in some mysterious way, and the flames running up the elevator-shaft soon set the upper stories in a blaze. The first and fourth floors were vacant, and the occupants of the second and third stories escaped, although with difficulty, as the halls were filled with smoke, but the fifth story was cut off by the dense smoke in the halls and the appearance of fire about the staircase before the alarm could be given. This story was occupied by a lady named Westlake, with her daughter and two young sons and an old servant. There were no fire-escapes on the building, and, waking to find their retreat by the stairs intercepted, they could do nothing but stand at the windows imploring help from the passers-by. An engine was quickly brought, but a stream from a hose was useless for saving life, and while a message was despatched for long ladders the foreman of the engine company undertook to reach the prisoners with a short scaling ladder. He managed to climb to the fifth story, followed by two or three other men, who waited at the third floor for his return. The brave fireman took one of the Westlake boys in his arms and climbed down with him to the fourth story, when he was overpowered by a rush of dense smoke from the windows and lost his hold of the ladder. His men below, seeing him about to fall, spread a life-net to catch him, and he and the child both fell into it, but rebounded from it to the pavement and were both severely injured. The firemen who had been waiting in the third story then jumped from the windows into the net and escaped unhurt. The remaining son and the daughter next tried this forlorn chance for their lives. The young lady dropped first from the window-sill to which she had been clinging, but struck a ladder before reaching the net, and was so badly injured that she died almost immediately. The boy, perhaps more fortunate, reached the net, but fell out again and was severely bruised on the stones. By this time the long ladders arrived and were quickly set up for the rescue of the mother, who was seen partly hanging out of a window. Before the firemen could reach her, however, she fell to the pavement and was picked up dead, suffocation by smoke having, however, been nearly complete before her fall.

THE architectural world is looking out rather anxiously for the announcement of the great competition for designs for the Palace of Congress at Buenos Ayres. Some time ago the Argentine Government appropriated six million dollars for the construction of this building, and a semi-official announcement was made that architects would be invited to submit designs in competition, but so far no formal publication of the terms has been made. It is understood, however, that eight months from the official announcement will be allowed for the completion of the designs and delivery at Buenos Ayres; or forty days less, if competitors prefer to deliver them at the nearest Argentine embassy. The jury is to be composed of the Presidents of the two Chambers, the Director and Vice-Director of the Department of Civil Engineering, the chief magistrate of the city of Buenos Ayres, and ten other members, to be appointed by the President, of whom five are to be architects. Six prizes are offered. The first prize is twenty thousand dollars, but five thousand of this will be retained until the author of the plan so honored has executed and delivered the working drawings of the building. The second prize is eight thousand dollars, the third, four thousand; and there are three inferior prizes, of two thousand dollars each.

IT seems rather curious to hear the name of Leonardo da Vinci invoked as the inventor of the device by which the Panama Canal is to be carried to completion, and made available for use, but there is no doubt that this great artist was the originator of the system of locks with movable gates now in use on nearly all canals and canalized rivers, and to this system the Panama Canal is now to be adapted. Although the change or the original scheme of a sea-level cutting to one including several locks was officially considered a year or more ago, it was only formally adopted this winter, yet such rapid progress has been made with the application of the lock system that contracts have already been entered into with M. Eiffel, the great engineer-builder, for the construction of the gates and other apparatus for eight enormous locks, all of the same pattern, by which vessels will be lifted from the Atlantic to the heights of the Culebra, and let down again to the Pacific. The gates, which form the most important part of the lock, are to be of iron, sliding in a groove, and the water is to be introduced and withdrawn from the lock by means of huge pipes, nine feet in diameter, which will pour ten million gallons of water into the lock basin in fifteen minutes.

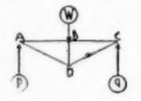
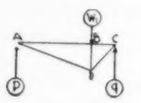
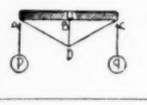
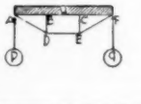
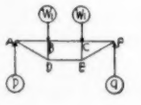
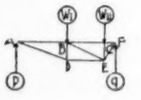
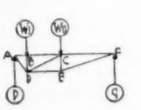
SAFE BUILDING.—XXIV.[1]

STRENGTH is frequently added to a girder or beam by trussing it, as shown in Table XVIII. One or two struts are placed against the lower (or upper) edge of a beam and a rod passed over them and secured to each end of the beam; by stretching this rod the beam becomes the compression chord of a truss and also a continuous girder running over one or two supports. There must therefore be enough material in the beam to stand the compression, and *in addition* to this enough to stand the transverse strains on the continuous girder. If the loads are concen-

Trussed Beams.

TABLE XVIII.

TRUSSED BEAMS.

Illustrations.	Description.	Compression in Struts.	Compression in Beam.	Tension in Rods.	Amount of Reactions.
	*Trussed Beam* with one centre load $= w$ $A\,B = B\,C$	*Compression in* $B\,D$ $= + w$	*Compression in* $A\,B$ $= + \frac{w}{2} \cdot \frac{A\,B}{B\,D}$ *Compression in* $B\,C$ same as in $A\,B$	*Tension in* $A\,D$ $= - \frac{w}{2} \cdot \frac{A\,D}{B\,D}$ *Tension in* $C\,D$ same as in $A\,D$	$p = \frac{w}{2}$ $q = \frac{w}{2}$
	*Trussed Beam* with one load $= w_{\text{I}}$ (not central,) at any point $A\,B \lesseqgtr B\,C$	*Compression in* $B\,D$ $= + w_{\text{I}}$	*Compression in* $A\,B$ $= + p \cdot \frac{A\,B}{B\,D}$ *Compression in* $B\,C$ same as in $A\,B$, or $= + q \cdot \frac{B\,C}{B\,D}$	*Tension in* $A\,D$ $= - p \cdot \frac{A\,D}{B\,D}$ *Tension in* $C\,D$ $= - q \cdot \frac{C\,D}{B\,D}$	$p = w_{\text{I}} \cdot \frac{B\,C}{A\,C}$ $q = w_{\text{I}} \cdot \frac{A\,B}{A\,C}$
	*Trussed Beam* with uniform load $= u$ and one central strut. $A\,B = B\,C$	*Compression in* $B\,D$ $= + \frac{5}{8} \cdot u$	*Compression in* $A\,B$ $= + \frac{5}{16} \cdot u \cdot \frac{A\,B}{B\,D}$ *Compression in* $B\,C$ same as in $A\,B$	*Tension in* $A\,D$ $= - \frac{5}{16} \cdot u \cdot \frac{A\,D}{B\,D}$ *Tension in* $C\,D$ same as in $A\,D$	$p = \frac{u}{2}$ $q = \frac{u}{2}$
	*Trussed Beam* with uniform load $= u$ and two struts, dividing beam into three equal parts. $A\,B = B\,C = C\,F$	*Compression in* $B\,D$ $= + \frac{11}{30} \cdot u$ *Compression in* $C\,E$ same as in $B\,D$	*Compression in* $A\,B$ $= + \frac{11}{30} \cdot u \cdot \frac{A\,B}{B\,D}$ *Compression in* $B\,C$ *and* *Compression in* $C\,F$ same as in $A\,B$	*Tension in* $A\,D$ $= - \frac{11}{30} \cdot u \cdot \frac{A\,D}{B\,D}$ *Tension in* $D\,E$ $=$ — compression in $A\,B$ *Tension in* $F\,E$ same as in $A\,D$	$p = \frac{u}{2}$ $q = \frac{u}{2}$
	*Trussed Beam* with two equal loads each $= w_{\text{I}}$ and two struts at equal distances from ends. $A\,B = C\,F$	*Compression in* $B\,D$ $= + w_{\text{I}}$ *Compression in* $C\,E$ same as in $B\,D$	*Compression in* $A\,B$ $= + w_{\text{I}} \cdot \frac{A\,B}{B\,D}$ *Compression in* $B\,C$ *and* *Compression in* $C\,F$ same as in $A\,B$	*Tension in* $A\,D$ $= - w_{\text{I}} \cdot \frac{A\,D}{B\,D}$ *Tension in* $D\,E$ $=$ — compression in $A\,B$ *Tension in* $F\,E$ same as in $A\,D$	$p = w_{\text{I}}$ $q = w_{\text{I}}$
	*Trussed Beam* with two unequal loads w_{I} and w_{II} at any points. Providing p smaller than w_{I} and q larger than w_{II}	*Compression in* $B\,D$ $= + p$ *Compression in* $B\,E$ $= + (q - w_{\text{II}}) \cdot \frac{B\,E}{B\,D}$ *Compression in* $C\,E$ $= + w_{\text{II}}$	*Compression in* $A\,B$ $= + p \cdot \frac{A\,B}{B\,D}$ *Compression in* $B\,C$ same as in $C\,F$ *Compression in* $C\,F$ $= + q \cdot \frac{C\,F}{B\,D}$	*Tension in* $A\,D$ $= - p \cdot \frac{A\,D}{B\,D}$ *Tension in* $D\,E$ $=$ — compression in $A\,B$ *Tension in* $F\,E$ $= - q \cdot \frac{F\,E}{B\,D}$	$p = \frac{w_{\text{I}} \cdot B\,F + w_{\text{II}} \cdot C\,F}{A\,F}$ $q = \frac{w_{\text{I}} \cdot A\,B + w_{\text{II}} \cdot A\,C}{A\,F}$
	*Trussed Beam* with two unequal loads w_{I} and w_{II} at any points. Providing p larger than w_{I} and q smaller than w_{II}	*Compression in* $B\,D$ $= + w_{\text{I}}$ *Compression in* $C\,D$ $= + (p - w_{\text{I}}) \cdot \frac{C\,D}{B\,D}$ *Compression in* $C\,E$ $= + q$	*Compression in* $A\,B$ $= + p \cdot \frac{A\,B}{B\,D}$ *Compression in* $B\,C$ same as in $A\,B$ *Compression in* $C\,F$ $= + q \cdot \frac{C\,F}{B\,D}$	*Tension in* $A\,D$ $= - p \cdot \frac{A\,D}{B\,D}$ *Tension in* $D\,E$ $=$ — compression in $C\,F$ *Tension in* $F\,E$ $= - q \cdot \frac{F\,E}{B\,D}$	$p = \frac{w_{\text{I}} \cdot B\,F + w_{\text{II}} \cdot C\,F}{A\,F}$ $q = \frac{w_{\text{I}} \cdot A\,B + w_{\text{II}} \cdot A\,C}{A\,F}$

Where p = the amount of the left reaction, in pounds.
" q = the amount of right reaction, in pounds.
" w, w_{I}, w_{II} = concentrated loads, in pounds.
" u = uniform load, in pounds, over whole beam.
" $A\,B$, $B\,C$, $C\,F$, $B\,D$, $B\,E$, $C\,D$, $C\,E$, $A\,D$, $D\,E$, $F\,E$ = the length of *longitudinal central* axes of these pieces, and must *all* be expressed *uniformly*, that is *all* expressed either in feet or inches.

The amounts of compression in either struts or beam — parts will be the total compression in each, expressed in pounds; to obtain the compression *per square inch*, divide the amount by the area of cross-section of the strut or part.

The amounts of tension in rods will be the total tension in each part, expressed in pounds; to obtain the tension *per square inch*, divide the amount by the area of cross-section of rod.

GLOSSARY OF SYMBOLS.—The following letters, *in all cases*, will be found to express the same meaning, *unless distinctly otherwise stated*, viz.: —

a = *area*, in square inches.
b = *breadth*, in inches.
c = constant for *ultimate resistance to compression*, in pounds, per square inch.
d = *depth*, in inches.
e = constant for *modulus of elasticity*, in pounds-inch, that is, pounds per square inch.
f = *factor-of-safety*.
g = constant for *ultimate resistance to shearing*, per square inch, across the grain.
g_{I} = constant for *ultimate resistance to shearing*, per square inch, lengthwise of the grain.
h = *height*, in inches.
i = *moment of inertia*, in inches. [See Table I.]
k = *ultimate modulus of rupture*, in pounds, per square inch.
l = *length*, in inches.
m = *moment or bending moment*, in pounds-inch.
n = *constant* in Rankine's formula for compression of long pillars. [See Table I.]
o = the *centre*.
p = the *amount* of the *left-hand re-action* (or support) of beams, in pounds.
q = the *amount* of the *right-hand re-action* (or support) of beams, in pounds.
r = *moment of resistance*, in inches. [See Table I.]
s = *strain*, in pounds.
t = constant for *ultimate resistance to tension*, in pounds, per square inch.
u = *uniform load*, in pounds.
v = *stress*, in pounds.
w = *load at centre*, in pounds.
x, y and z signify *unknown quantities*, either in pounds or inches.
δ = *total deflection*, in inches.
ρ^2 = *square of the radius of gyration*, in inches. [See Table I.]
$\mathfrak{d}$ = *diameter*, in inches.
$\mathfrak{r}$ = *radius*, in inches
π = 3.14159, or, say, 3 1-7 signifies the *ratio of the circumference and diameter of a circle.*

If there are more than one of each kind, the second, third, etc., are indicated with the Roman numerals, as, for instance, a, a_{I}, a_{II}, a_{III}, etc., or b, b_{I}, b_{II}, b_{III}, etc.

In taking moments, or bending moments, strains, stresses, etc., to signify at what point they are taken, the letter signifying that point is added, as, for instance: —

m = moment or bending moment at *centre*.
m_{A} = " " " *point A*.
m_{B} = " " " *point B*.
m_{X} = " " " *point X*.
s = strain at *centre*.
s_{B} = " *point B*.
s_{X} = " *point X*.
v = stress at *centre*.
v_{D} = " *point D*.
v_{X} = " *point X*.
w = load at *centre*.
w_{A} = " *point A*

[1] Continued from No. 636, page 108.

trated immediately over the braces, there will be no transverse strain whatever, but the braces will be compressed the full amount of the respective loads on each. In the case of uniform loads, transverse strains cannot be avoided, of course, but where loads are concentrated the struts should always be placed immediately under them. Even where loads are placed very unevenly, it is better to have the panels of the truss irregular, thus avoiding cross or transverse strains. This same rule holds good in designing trusses of any kind.

Struts placed under load.

Table XVIII shows very clearly the amount and kind of strains in each part of trussed beams. Where there are two struts and they are of any length care must be taken by diagonal braces or otherwise, to keep the lower ends of braces from tipping towards each other. Theoretically they cannot tip, but practically, sometimes, they do. Care must be taken that the beam is braced sideways, or else it must be figured for its safety against lateral flexure (Formula 5.) Then it must have material enough not to shear off at supports, nor to crush its under side where lying on support. The ends of rods must have sufficient bearing not to crush the wood. Iron shoes are sometimes used, but if very large are apt to rot the wood. In that case it is well to have a few small holes in the shoes, to allow ventilation to end of timber. If iron straps and bolts are used at the end, care must be taken that the strap does not tear apart at bolt holes; that it does not crush itself against bolts; that it does not shear off the bolts, and that it does not crush in the end of timber. Care must also be taken to have enough bolts, so that they do not crush the wood before them, and to keep the bolts from shearing out, that is tearing out the wood before them. In all trusses and trussed works the joints must be carefully designed to cover all these points. Many architects give tremendous sizes for timbers and rods in trusses, thus adding unnecessary weight, but when it comes to the joint, they overlook it, and then are surprised when the truss gives out. The next time they add more timber and more iron, till they learn the lesson. It must be remembered that the strength of a truss is only equal to the strength of its weakest part, be that part a member or only a part of a joint. This subject will be fully dealt with in the chapter on Trusses.

Necessary Conditions.

Importance of Joints.

The deeper the truss is made, that is, the further we separate the top and bottom chords, the stronger will it be; besides additional depth adds very much to the stiffness of a truss.

Depth Desirable.

All trussed beams, and all trusses should be "cambered up," that is, built up above their natural lines sufficiently to allow for settling back into their correct lines, when loaded. The amount of the camber should equal the calculated deflection. For all beams, girders, etc., of uniform cross-section throughout, the deflection can be calculated from Formulæ (37) to (42) according to the manner of loading. For wrought-iron beams and plate-girders of uniform cross-section throughout, the deflection can be calculated from the same formulæ; where, however, the load is uniform and it is desired to simplify the calculation, the deflection can be quite closely calculated from the following Formula:

Deflection of Girders and Beams.

Uniform Cross-section and Load.

$$\delta = \frac{L^2}{75.d} \qquad (79)$$

Where δ = the greatest deflection at centre, in inches, of a wrought-iron beam or plate girder of uniform cross-section throughout, and carrying its total safe uniform load, calculated for rupture only.

Where L = the length of span, in feet.

Where d = the depth of beam or girder in inches.

If beam or plate girder is of steel, use $64\frac{1}{2}$ instead of 75.

If the load is not uniform, change the result, as provided in cases (1) to (8), Table VII.

For a centre load we should use $93\frac{3}{4}$ in place of 75 or

Uniform Cross-section, Centre Load.

$$\delta = \frac{L^2}{93\frac{3}{4}.d} \qquad (80)$$

Where values are the same, as for Formula (79) except that beam or girder carries its total safe centre load, calculated for rupture only.

If beam or girder is of steel use $80\frac{3}{4}$ instead of $93\frac{3}{4}$.

Therefore not to crack plastering and yet to carry their full safe loads, wrought-iron beams or plate girders should never exceed in length (measured in feet) twice and a quarter times the depth (measured in inches), if the load is uniform, or

Safe length, uniform Cross-section and Load.

$$2\tfrac{1}{4}.d = L \qquad (81)$$

Where L = the ultimate length of span (not to crack plastering), in feet, of a wrought-iron beam or plate girder, of uniform cross-section throughout and uniformly loaded with its total safe load.

Where d = the depth of beam or girder in inches.

If beam or girder is of steel, use 2 instead of $2\frac{1}{4}$.

If the load is central the length in feet should not exceed $2\frac{1}{8}$ times the depth in inches, or

Safe length, uniform Cross-section, Centre Load.

$$2\tfrac{1}{4}.d = L. \qquad (82)$$

Where L = the ultimate length of span in feet (not to crack plastering), of a wrought-iron beam or plate girder, of uniform cross-section throughout, and loaded at its centre with its total safe load.

Where d = the depth of beam or girder in inches.

If beam or girder is of steel use $2\frac{2}{3}$ instead $2\frac{1}{4}$.

Deepest beam most economical.

One thing should always be remembered, when using iron beams, and that is, that the deepest beam is *always* not only the stiffest, but the most economical. For instance, if we find it necessary to use a $10\frac{1}{2}''$ beam — 105 pounds per yard, it will be cheaper to use instead the 12′ beam — 96 pounds per yard. The latter beam not only weighs 9 pounds per yard less, but it will carry more, and deflect less, owing to its extra two inches of depth. This same rule holds good for nearly all sections.

Deflection of Trusses.

To obtain the deflections of trussed beams or girders by the rules already given would be very complicated. For these cases, however, Box gives an approximate rule, which answers every purpose. He calculates the amount of extension in the tension (usually the lower) chord, and the amount of contraction in the compression (usually the upper) chord, due to the strains in each, and from these, obtains the deflections. Of course the *average* strain in each chord must be taken and not the greatest strain at any one point in either. In a truss, where each part is proportioned in size to resist exactly the compressive or tensional strain on the part, every part will, of course, be strained alike; the strain in the compressive member being $= \left(\frac{c}{f}\right)$ per square inch, throughout the whole length, and in the tension member $= \left(\frac{t}{f}\right)$ per square inch, throughout the whole length.

The same holds good for plate girders, where the top and bottom flanges are diminished towards the ends, in proportion to the bending moment. But where, as in wrought-iron beams (and in many trusses), the flanges are made, for the sake of convenience, of uniform cross-section throughout their entire length, the "average" strain will, of course, be much less, and consequently the beam or girder stiffer.

Average Strain in Chords.

If we construct the graphical representation of the bending moments at each point of beam (as will be explained in the next Chapter) and divide the area of this figure in inch-pounds by the length of span in inches, we will obtain the average strain in either flange, provided the flange is of uniform cross-section throughout, or

Uniform Cross-section.

$$v = \frac{a}{l} \qquad (83)$$

Where v = the *average* strain, in pounds, on top or bottom flange or chord, where beam or girder is of uniform cross-section throughout.

Where l = the length of span, in inches.

Where a = the area in pounds-inch of the graphical figure giving the bending moment at all points of beam.

To obtain the dimensions of this figure measure its base line (or horizontal measurement) in inches, and its height (or vertical measurement) in pounds, assuming the greatest vertical measurement as $= \left(\frac{c}{f}\right)$ or $= \left(\frac{t}{f}\right)$, in pounds, according to which flange we are examining.

Thus, in the case of a uniform load, this figure would be a parabola, with a base of length equal to the span measured in inches, and a height equal to the greatest fibre strains in pounds; the average

strain therefore in the compression member of a beam, girder or truss, of uniform cross-section throughout would be, — (remembering that the area of a parabola is equal to two-thirds of the product of its height into its base),

Uniform load and Cross-section.

$$v = \frac{\frac{2}{3} . l . \left(\frac{c}{f}\right)}{l} \text{ or}$$

$$v = \frac{2}{3} . \left(\frac{c}{f}\right) \qquad (84)$$

Where v = the *average* strain, in pounds, in compression flange or chord of a beam, girder or truss of uniform cross-section throughout and carrying its total safe uniform load.

Where $\left(\frac{c}{f}\right)$ = the safe resistance to compression per square inch of the material.

It is supposed, of course, that at the point of greatest bending moment — or where the greatest compression strain exists — that the part is designed to resist or exert a stress = $\left(\frac{c}{f}\right)$ per square inch. If the greatest compression stress is less, insert its value in place of $\left(\frac{c}{f}\right)$. Of course, it must never be greater than $\left(\frac{c}{f}\right)$.

Similarly we should have

Uniform Load and Cross-section.

$$v = \frac{2}{3} . \left(\frac{t}{f}\right) \qquad (85)$$

Where v = the *average* strain, in pounds, in tension flange or chord of a beam, girder or truss of uniform cross-section throughout, and carrying its total safe uniform load.

Where $\left(\frac{t}{f}\right)$ = the safe resistance to tension, per square inch, of the material.

It being understood that at the point of greatest bending moment — or where the greatest tension strain exists — that the part is designed to resist or exert a stress = $\left(\frac{t}{f}\right)$ per square inch. If this greatest tensional stress is less than $\left(\frac{t}{f}\right)$ insert its value in its place in Formula (85). Of course, it must never be greater than $\left(\frac{t}{f}\right)$.

For a beam, girder or truss with a load concentrated at the centre, but with flanges or chords of uniform cross-section throughout, the average strain would be just one-half that at the centre; for, the bending-moment graphical-figure will be a triangle, and inserting the values in Formula (83) would give for the compression member:

Centre Load Uniform Cross-Section.

$$v = \tfrac{1}{2} . \left(\frac{c}{f}\right) \qquad (86)$$

and for the tension member:

$$v = \tfrac{1}{2} . \left(\frac{t}{f}\right) \qquad (87)$$

The meaning of letters being the same as in Formulæ (84) and (85), but the total safe load being concentrated at the centre instead of uniformly distributed.

To obtain the amount of contraction or expansion due to this average strain, use the following Formula:

Expansion or Contraction from Strain.

$$x = \frac{v . l}{e} \qquad (88)$$

Where v = the *average* strain, in pounds per square inch, in either chord or flange.

Where l = the length of span, in inches.

Where e = the modulus of elasticity of the material, in pounds-inch.

Where x = the amount of extension or contraction, in inches, of the chord or flange.

Now let us apply the above rules to beams, plate girders, and trussed beams. Taking the case of a beam or plate girder or truss with parallel flanges or chords.

Figure 145 shows the same, after the deflection has taken place. We can now assume approximately, that $C\,A$ is equal to one-half the difference between the contraction of $G\,C$ and the elongation of $H\,B$, or, what amounts to the same thing, that $C\,A$ is equal to one-half the sum of the contraction of the one and the elongation of the other.

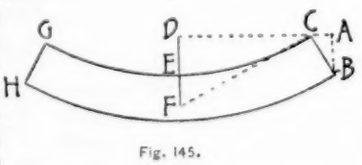


Fig. 145.

Further, we can assume that approximately, $A\,B = d$ or the depth of beam, and $C\,D = \frac{l}{2}$ or one half the span.

The curve $C\,E\,C$ will approximate a parabola, so that if we draw a tangent $C\,F$ to the same at C, we know that $D\,E = E\,F = \frac{D\,F}{2}$ or $D\,F = 2 . D\,E$. But as $D\,E$ represents the deflection (δ) of the beam, we have

$$D\,F = 2 . \delta .$$

Now as $C\,F$ is normal to $C\,B$, and $C\,D$ normal to $A\,B$, we know that angles $D\,C\,F = A\,B\,C$; further, as both triangles are right angle triangles, we know that they are similar, therefore:

$$D\,F : C\,A :: D\,C : A\,B, \text{ or}$$

$$2 . \delta : C\,A :: \frac{l}{2} : d \text{ or}$$

$$\delta = \frac{C\,A . \frac{l}{2}}{2 . d} = \frac{C\,A . l}{4 . d .}$$

If now we assume the sum of the extension and contraction of the two flanges or chords to be $= x$.

We have $C\,A = \frac{x}{2}$ or

Deflection of Parallel Flanges or Chords, any Cross-section.

$$\delta = \frac{x . l}{8 . d} \qquad (89)$$

Where δ = the deflection, in inches, of a beam, plate girder or truss, with parallel flanges or chords.

Where x = the sum of the amount of extension in tension chord, plus the amount of contraction in compression chord.

Where l = the length of span, in inches.

Where d = the total depth of beam, girder or truss in inches.

Take the case of a wrought-iron plate girder or beam of uniform cross-section throughout carrying its full uniform load, we should have the strain at the centre on the extreme fibres = 12000 pounds per square inch. Now the average strain on both upper and lower flanges would be, Formulæ (84) and (85).

$$v = \tfrac{2}{3} . \quad 12000 = 8000 \text{ pounds}$$

per square inch. Therefore amount of contraction in upper flange Formula (88), (and remembering that, from Table IV, $e = 27000000$)

$$x = \frac{8000 . l}{27000000} = \frac{l}{3375}$$

The elongation of the bottom flange would be an equal amount, therefore the sum of the two

$$x_1 = 2 . x = \frac{2 . l}{3375}$$

$$= \frac{l}{1687,5}$$

Inserting these values in Formula (89) we have the deflection

$$\delta = \frac{l^2}{8 . 1687,5 . d} = \frac{l^2}{13500 . d}$$

and inserting for $l^2 = 144 . L^2$, we have

$$\delta = \frac{144 . L^2}{13500 . d .}$$

$$= \frac{L^2}{93\frac{3}{4} . d .}$$

Had we assumed that the area of flanges or chords diminished towards the supports in proportion to the bending moment or actual stresses required, the average strain would, of course, be 12000 pounds per square inch throughout the entire length, no matter how the load might be applied.

Inserting this value in Formula (88) we should have had, for the amount of contraction of top flange

$$x = \frac{12000 . l}{27000000} = \frac{l}{2250}$$

The same for the extension of bottom chord, or

$$x_1 = 2 . \frac{l}{2250} = \frac{l}{1125}$$

Inserting these in Formula (89) we have for the deflection:

$$\delta = \frac{l^2}{8 . 1125 . d} = \frac{l^2}{9000 . d}$$

Inserting $144\ L^2 = l^2$ we have

$$\delta = \frac{144 . L^2}{9000 . d} \text{ or}$$

Parallel flanges or Chords, Diminished Cross-section, any loads.

$$\delta = \frac{L^2}{62\frac{1}{2}.d} \qquad (90)$$

Where δ = the greatest deflection, in inches, of a wrought-iron plate girder, or wrought-iron truss, with parallel flanges or chords, and where the areas of flanges or chords are gradually diminished towards supports, and no matter how the load is applied; in no part however must the stresses, per square inch exceed respectively either $\left(\frac{c}{f}\right)$ or $\left(\frac{t}{f}\right)$.

Where L = the length of span, in feet.

Where d = the total depth (heights), in inches, from top of top flange or chord to bottom of bottom flange or chord.

If girder or truss is of steel, use $53\frac{1}{3}$ instead of $62\frac{1}{2}$.

From Formula (90) and Formula (28) we get the rule that (no matter how the load is applied) if we want to carry the full safe load and not have deflection enough to crack plastering the length in feet must not exceed $1\frac{7}{8}$ times the total depth in inches.

For:

$$L.\ 0{,}03 = \frac{L^2}{62\frac{1}{2}.d} \text{ or}$$

$$L = 62\tfrac{1}{2}.\,0{,}03.\,d$$

$$= 1{,}875.\,d \text{ or say}$$

Safe Length, Diminished Cross-section, any Load, Parallel Flanges or Chords.

$$L = 1\tfrac{7}{8}.d \qquad (91)$$

Where L = the length, in feet, of a wrought-iron plate girder or wrought-iron truss, with parallel flanges or chords and with area of flanges or chords diminishing gradually towards supports and no matter how the load is applied; in no part however must the stresses, per square inch, exceed respectively either $\left(\frac{c}{f}\right)$ or $\left(\frac{t}{f}\right)$.

Where d = the total depth (height), in inches, from top of top flange or chord to bottom of bottom flange or chord.

If girder or truss is of steel, use $1\frac{7}{8}$ instead of $1\frac{7}{8}$.

We see therefore that a beam of diminishing cross-section throughout is only about $\frac{3}{4}$ as stiff, as one with uniform cross-section, as its amount of deflection will be one-half more than that of the latter. Both deflections are approximate only, however, as we see by comparing the amount for the uniform cross-section to that obtained from Formula (79). The deflection for varying cross-sections however can be assumed as nearly enough correct, as these are never diminished so much practically as we have assumed in theory. Now taking the case of a trussed beam.

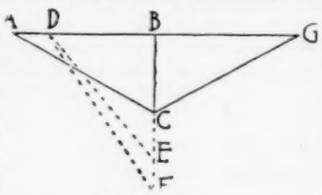


Fig. 146.

Deflection Trussed Beam. In Figure 146, let $A\ B$ be one half of a trussed beam, let $B\ C$ be the strut and $A\ C$ the tie. We will consider the load concentrated at B. Now the first effect is to shorten $A\ B$ by compression, let us say to $D\ B$.

Then, of course, $A\ D$ will represent one half of the contraction in the whole beam $A\ G$. Now the end of rod A moving to D will, of course, let the point C down to E, if we make $D\ E = A\ C$.

But there will be an elongation in $D\ E$ besides, due to the tension in it, which will let it down still further, say to F, if $D\ F = A\ C +$ elongation in $A\ C$, of course the point B will move down too, but we can overlook this to avoid complication. We now have $C\ F$ representing the amount of the deflection. To this should be added the amount of contraction of $B\ C$ due to the compression in it. We can readily find $C\ F$.

We know that

$$B\ F = \sqrt[2]{D\ F^2 - D\ B^2}$$

Now $D\ F$ we know is $= A\ C$ plus the elongation of $A\ C$ due to the tension in it, which we can find from Formula (88). From same formula we find the amount of contraction in $A\ G$ of which $A\ D$ is one-half, subtracting this from $A\ B$ or $\frac{l}{2}$ leaves, of course, $D\ B$.

Now having found $B\ F$ we substract from it $B\ C$, the length of which is known, and the balance is of course the deflection $C\ F$; to this we add the contraction of $B\ C$ and obtain the total deflection of the whole trussed beam.

If the load had been a uniform load, instead of a concentrated one over the strut, there would be a deflection in that part of $A\ G$ which would be acting as a continuous girder. But this deflection would take place between B and G and between B and A and would not affect the deflection of the whole trussed beam.

An example will make much of the foregoing more clear.

Example.

Trussed Beam. *A trussed Georgia Pine beam is 16″ deep and of 24 feet clear span; it bears 16″ on each support and is trussed as shown in Figure 147. The beam carries a uniformly distributed load of 40800 pounds on the whole span, including weight of beam and trussing. Of what size should the parts be?*

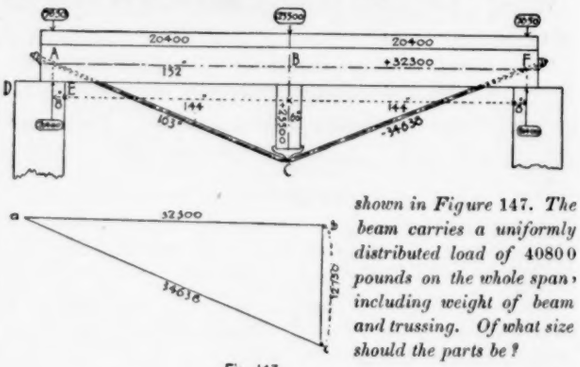


Fig. 147.

We draw the longitudinal neutral axes of each part, namely $A\ B$ $B\ C$ and $A\ C$. The latter is so drawn that the neutral axis of the reaction, which is of course half way between D and E (or 8″ from E) will also pass through A.

In designing trusses this should always be borne in mind, that so far as possible all the neutral axes at each joint should go through the same point.

Cross-strains in Beams. The beam $A\ F$ virtually becomes a continuous girder, of two equal spans of 12 feet or 144″ each, uniformly loaded with 20400 pounds each, and supported at three points A, B and F. From table XVII we know that the greatest bending moment is at B and

$$= \frac{u.l}{8} = \frac{20400.144}{8} = 367200 \text{ pounds-inch.}$$

The modulus of rupture for Georgia pine (Table IV) is $\left(\frac{k}{f}\right) = 1200$, therefore moment of resistance (r) from Formula (18) and Table I, section No. 2,

$$r = \frac{b.d^2}{6} = \frac{367200}{1200} \text{ or}$$

$$b.d^2 = 1836$$

Now we know that $d = 16$, or $d^2 = 256$, therefore

$$b = \frac{1836}{256} = 7{,}2 \text{ or say we need a beam } 7\tfrac{1}{4}\text{″ x 16″ for the}$$

transverse strain. We must add to this however for the additional compression due to the trussing.

Compression in Strut. The amount of the load carried by strut $C\ B$, see Table XVII, is

$= \frac{5}{8}.u$ from each side, or

$= 25500$ on the strut $B\ C$, of which

$= 12250$ from each side.

Compression in Beam. If now we make at any scale a vertical line $b\ c = half$ the load carried at point B or $= 12250$ in our case, and draw $b\ a$ horizontally and $a\ c$ parallel to $A\ C$, we find the strain in $B\ A$ by measuring $b\ a = (32300$ pounds) or in $A\ C$ by measuring $a\ c = (34638$ pounds) both measured at same scale as $b\ c$. We find, further, in passing around the triangle $c\ b\ a\ c$ — ($c\ b$ being the direction of the reaction at A), that $b\ a$ is pushing towards A, therefore compression; and that $a\ c$ is pulling away from A, therefore tension. Using the usual signs of + for compression, and — for tension, we have then:

$A\ B = +\ 32300$ pounds.

$A\ C = -\ 34638$ pounds.

$B\ C = +\ 25500$ pounds.

Had we used Table XVIII we should have had the same result for:

Compression in $A\,B = \frac{25500}{2} \cdot \frac{A\,B}{B\,C} = +32300$ pounds and

Tension in $A\,C = \frac{25500}{2} \cdot \frac{A\,C}{B\,C} = -34638$ pounds.

Now the safe resistance of Georgia pine to compression along fibres (Table IV) is

$$\left(\frac{c}{f}\right) = 750 \text{ pounds.}$$

If $A\,B$ were very long, or the beam very shallow or very thin, we should still further reduce $\left(\frac{c}{f}\right)$ by using Formula (3). But we can readily see that the beam will not bend much by vertical flexure due to compression, nor will it deflect laterally very much, so we can safely allow the maximum safe stress per square inch, or 750 pounds and consider $A\,B$ a short column.

The necessary area to resist the compression, Formula (2) is:

$$32300 = a.\,750 \text{ or}$$

$$a = \frac{32300}{750} = 43 \text{ square inches.}$$

As the beam is 16″ deep, this would mean an additional thickness $= \frac{4}{16} = 2\frac{1}{16}$

Adding this to the $7\frac{1}{4}$″ already found to be necessary, we have

$$7\tfrac{1}{4} + 2\tfrac{1}{16} = 9\tfrac{5}{16}$$

or the beam would need to be, say 10″ x 16″.

Size of Strut. Now the size of $B\,C$ must be made sufficient not to crush in the soft underside of the beam at B. The bearing here would be across the fibres of the beam, and we find (Table IV) that the safe compressive stress of Georgia pine across the fibres is $\left(\frac{c}{f}\right)$ = 200 pounds. We need therefore an area

$$a = \frac{25500}{200} = 128 \text{ inches.}$$

As the beam is only 10″ wide the strut $B\,C$ will have to measure, $\frac{128}{10} = 12\frac{4}{5}$ inches the other way, or we will say it could be 10″ x 12″. This strut itself might be made of softer wood than Georgia pine, say of spruce; the average compression on it is

$$\frac{25500}{10.12} = 212 \text{ pounds per square inch.}$$

Now spruce will stand a compression on end (Table IV) of $\left(\frac{c}{f}\right)$ = 650 or, even if spruce is used, the actual strain would be less than one-third of the safe stress. At the foot of the strut $B\,C$ we put an iron plate, to prevent the rod from crushing in the wood. The rod itself must bear on the plate at least

Iron Shoe to Strut. $\frac{25500}{12000} = 2{,}1$ square inches, or it would crush the iron — (12000 pounds being the safe resistance of wrought-iron to crushing).

Size of Tie-rod. The safe tensional stress of wrought-iron being 12000 pounds per square inch (Table IV), we have the necessary area for tie-rod $A\,C$ from Formula (6)

$$34638 = a\,.\,12000 \text{ or}$$

$$a = \frac{34638}{12000} = 2{,}886 \text{ square inches.}$$

From a table of areas we find that we should require a rod of 1 15–16″ diameter, or say a 2″ rod.

The area of a 2″ rod being = 3,14 square inches the actual tensional stress, per square inch on the rod, will be only

$$\frac{34638}{3{,}14} = 11312 \text{ pounds per square inch.}$$

Size of Washer. We must now proportion the bearing of the washer at "A" end of tie-rod. The amount of the crushing coming on washer will be whichever of the two strains at A, (viz. $B\,A$ and $A\,C$) is the lesser, or $B\,A$ in our case, which is 32300 pounds. We must therefore have area enough to the washer not to crush the end of beam (or along its fibres), the safe resistance of which we already found to be: $\left(\frac{c}{f}\right) = 750$ pounds per square inch; we need therefore.

$$\frac{32300}{750} = 43 \text{ square inches.}$$

The washer therefore should be about

$$6\tfrac{1}{2}\text{″ by } 6\tfrac{1}{2}\text{″}$$

Upset Screw-end. The end of the rod *must* have an "upset" screw-end; that is, the threads are raised above the end of rod all around, so that the area at the bottom of sinkage, between two adjoining threads, is still equal to the full area of rod. If the end is not "upset" the whole rod will have to be made enough larger to allow for the cutting of the screw at the end, which would be a wilful extravagance.

It is unnecessary to calculate the size of nuts, heads, threads, etc., as, if these are made the regulation sizes, they are more than amply strong. **Central Swivel.** It should be remarked here that in all trussed beams, if there is not a central swivel, for tightening the rod, that there should be a nut at *each* end of the rod; and not a head at one end and a nut at the other. Otherwise in tightening the rod from one side only it is apt to tip the strut or crush it into the beam on side being tightened. We must still however calculate the vertical shearing across the beam at the supports, which we know equals the reaction, or 20400 pounds at each end. To resist this we have 10″ x 16″ = 160 square inches, less 3″ x 16″, cut out to allow rod end to pass, or say 112 square inches net, of Georgia pine, across the grain; and as $\left(\frac{g}{f}\right) = 570$ pounds per square inch (see Table IV); the safe vertical shearing stress at each support would be (Formula 7)

$$112.570 = 63840 \text{ pounds or more than three times the}$$

actual strain. **Bearing of Beam.** Then, too, we should see that the bearing of beam is not crushed. It bears on each reaction 16 inches, or has a bearing area = 16.10 = 160 square inches. $\left(\frac{c}{f}\right)$ for Georgia pine, across the fibres, Table IV, is

$$\left(\frac{c}{f}\right) = 200, \text{ therefore the beam will bear safely at each}$$

end

$$160.200 = 32000 \text{ pounds or about one-half more than the}$$

reaction. There will be no horizontal shearing, of course, except in that part of beam under transverse strain, and this certainly cannot amount to much. The beam is therefore amply safe.

Deflection of Beam. Now let us calculate the deflection. The modulus of elasticity for Georgia pine, Table IV is: $e = 1200000$ pounds-inch. The average compression strain in $A\,F$ was 750 pounds per square inch, therefore the amount of contraction (Formula 88)[1]

$$x = \frac{750.304}{1200000} = 0{,}19 \text{ inches.}$$

Now $A\,D$ (in Fig. 146) will be one-half of this, or 0,095 inches.

The amount of elongation in $A\,C$ will be, remembering that we found the average stress to be only 11312 pounds per square inch, and that for wrought-iron $e = 27000000$ (Formula 88)

$$x = \frac{11312.163}{27000000} = 0{,}0682$$

The exact length of $A\,C$ (Fig. 147 should be 163,41 not 163″).
Therefore $D\,F$ (Fig. 146) will be

$$D\,F = 163{,}41 + 0{,}0682 = 163{,}4782''$$

$$D\,B = 152 - 0{,}19$$

$$= 151'', 81$$

Therefore (Fig. 146)

$$B\,F = \sqrt[2]{163{,}4782^2 - 151{,}81^2}$$

$$= 60'', 655$$

Now $B\,C$ (Fig. 147) would be = 60″, deducting this from the above we should have a deflection = 0″, 655.

To this we must add the contraction of $B\,C$. The strut will be less than 60″ long, say about 50″. The average compressive stress per square inch we found = 212 pounds. The modulus of elasticity for spruce, Table IV, is $e = 850000$, therefore contraction in strut (Formula 88)

$$x = \frac{212.50}{850000} = 0{,}0125$$

Adding this to the above we should have the total deflection

$$\delta = 0{,}655 + 0{,}0125$$

$$= 0{,}6675$$

This would be the amount we should have to "camber" up the beam, or say 3–4″.

[1] In reality the contraction of $A\,F$ would be much less, as the part figured for transverse strain only would very materially help to resist the compression, one half of it being in tension.

The safe deflection not to crack plastering, would be (Formula 28)

$$\delta = L.\ 0{,}03$$
$$= 24.0{,}03$$
$$= 0{,}72$$

So that our trussed beam is amply stiff.

Louis DeCoppet Berg.

[To be continued.]

THE ILLUSTRATIONS

[Contributors are requested to send with their drawings full and adequate descriptions of the buildings, including a statement of cost.]

THE KENT GATE, QUEBEC, CANADA.

[Gelatine print, issued only with the Imperial Edition.]

This view is taken from without the city walls. The little Jesuits' chapel shows on the left.

UNITED STATES COURT-HOUSE AND POST-OFFICE, SAN ANTONIO, TEX. MR. W. A. FRERET, SUPERVISING ARCHITECT.

Some of the details shown here, together with those shown in the *American Architect* for February 4th, will serve to illustrate some parts of the papers in United States Government Building Practice.

COMPETITIVE DESIGN FOR THE CITY-HALL, MINNEAPOLIS, MINN. MESSRS. LONG & KEES, ARCHITECTS, MINNEAPOLIS, MINN.

To these gentlemen was awarded the fourth prize.

SOME OLD BELLS.

These drawings were made after some loose sheets of the *Allgemeine Bauzeitung* that came into our hands by chance some time ago, and as they had no title, we have not been able to identify the bells.

OLD BELLS.

A writer in *Chambers' Journal* gives an account of some old bells which perhaps will interest the readers of this journal:

The origin of the bell is not known; but a knowledge of it goes back to a period beyond the written history of nations. The pious Dionysius Barsalabi, in his dissertation on bells, asserts that he finds it recorded in several histories that Noah received a command that the workmen employed in building the Ark should be summoned to their labor by the strokes of wood on a bell; but the earliest mention of them in Scripture is found in Exodus xxviii, 33-35, and xxxix, 25, when speaking of the necessary ornaments for the hem of the high-priest's robe: "And beneath, upon the hem of it, thou shalt make pomegranates of blue and of purple and of scarlet, round about the hem thereof; and bells of gold between them round about; a golden bell and a pomegranate upon the hem of the robe round about. And it shall be upon Aaron to minister: and his sound shall be heard when he goeth in unto the holy place before the Lord, and when he cometh out, that he die not." "And they made bells of pure gold, and put the bells between the pomegranates upon the hem of the robe." It is possible that the Assyrians and Egyptians used

bells exclusively in religious rites, but the Greeks and Romans employed them for secular as well as for religious purposes. At the British Museum may be seen, in a case in the Nimroud Gallery of Assyrian antiquities, eighty small bronze bells with iron tongues, which were found by Layard in a caldron when excavating Nimroud — the ancient Calah of Scripture on the banks of the Tigris, the approximate date of which city may be given from B. C. 885 to 630. The great feasts of Osiris, the judge of the dead, were inaugurated by the Egyptian priests with the ringing of hand-bells, and the Greek priests of Cybele followed the same custom when they sacrificed to the "mother of a hundred gods." Later, they were in more general use with both Greeks and Romans. Pliny refers to the sounding of a bell in public places in Athens to advertise the sale of fish — doubtless the predecessor of the modern town-crier, who may still be heard in remote country districts. The Greek sentries in camps and garrisons, when they heard the ringing of a bell, knew the relief guards were approaching, and were bound to answer the signal. At Rome also the musical tinkling announcing the hour for the indulgence of the luxurious bath was welcomed by the Romans, who made great use of bells as personal ornaments, and adopted them for emblems on their triumphal processional cars.

The small quadrangular hand-bells, made of thin plates of hammered iron, riveted together at the sides and bronzed — a form represented on some of the old Irish stone crosses, and specimens of which are in the British and Hibernian museums — were exclusively used for ecclesiastical purposes. Their introduction into Britain is generally assigned to the wandering monks, who in those early days of Christianity made frequent pilgrimages to Italy. Ireland possesses a rich collection of those old bells, some of which, with a traditional history, are preserved in costly shrines, embellished with gems. In the "*Annals of the Four Masters*" mention is made of the "bell of St. Patrick," which has ever been held in special veneration because of the belief that it was the property of that saint. For generations this relic was in the possession of the Mulholland family, who kept it buried in order to insure its safety during the disturbances which so frequently troubled their country. The last descendant of the family bequeathed the treasure and the secret of its hiding-place to the late Adam McClean, who, on searching, duly found in the spot indicated a strong oaken box, containing the old bell, enclosed in its lovely shrine, and with a Bible written in early Irish characters. This bell is only six inches high, five broad and four deep; the shrine is of beaten brass, covered with an antique design of gold and silver filigree worked in complicated convolutions and knots. The whole is profusely studded over with rock crystals, garnets and other precious stones. It is now in the Royal Irish Academy — an interesting collection that includes the almost unique "bell of Armagh," besides others, rivals in age and beauty. But as a priceless specimen of the skill and workmanship of those early days, none of the caskets in which each bell is placed equals that of St. Patrick. Supplementary to these small bells, used in the services of the church, are others employed for the administration of oaths, which oaths were considered essentially binding and sacred. Apart from the veneration felt for these bells, superstition sometimes invested them with peculiar powers, like the "bell of St. Columbia," for example, known as "Dia Diagheltus" (God's vengeance), which the taker of the oath believed could inflict on a perjurer a terrible and indescribable punishment. Dr. Beresford, the late Archbishop of Armagh, had four very curious old bells of this class. The venerable prelate purchased them at different times and in different parts of Ireland from peasants, whose reverence for their sanctity had declined in these days of progress, and who, fortunately, were not unwilling to part with things to them comparatively worthless, but above all price to a collector.

The suspended bell is a recent introduction compared with the antiquity of the hand-bell used in heathen as in Christian times in the celebration of religious rites. The development of the heavy swinging bell, from the time of the Anglo-Saxons to that of the Normans, must have been tolerably rapid, when the great size and strength of the belfries built by the latter is considered. About the middle of the seventh century, in the reign of Egfrid, Benedict, Abbot of Wearmouth and of Jarrow-upon-Tyne, presented some large bells to his church; and about the same period the Venerable Bede relates how the nuns of St. Hilda at Whitby, were summoned to prayers by the sound of bells. At the present day very few bells are left bearing authentic dates previous to the Reformation, although it is said that one was removed from the belfry of an old church in Cornwall, inscribed "Alfredux Rex," which must, if the inscription was correct, have been in use for a thousand years.

Amoy. — Like most important Chinese cities, Amoy is encircled by a mighty wall, writes Miss Gordon Cumming in the *St. James's Gazette*. The summit of these walls invariably affords the most agreeable walk available; it is the only place which is never crowded, and here a good general view of the city can be obtained. To most Europeans the walls of Canton afford a general standard of size. Those of Amoy are considerably smaller, and in the city itself two features conspicuous in Canton are lacking; namely, the tall pagodas and the great square towers, which are the much-frequented pawn-shops. Descending from the walls one immediately enters a labyrinth of dirty streets and markets, with bewildering crowds forever hurrying to and fro, a thousand details of interest arresting one's attention at every turn. As a matter of course, any one wandering through a Chinese city enters temples innumerable; for though, for by far the most part, they are amazingly dirty, there are generally some distinctive features of interest to be noted. Among these I may mention a fine image of Kwan-yin, the thousand-armed Goddess of Mercy, the special feature being that the great golden halo within which she stands is (or was) formed of a thousand golden hands.

UNITED STATES GOVERNMENT BUILDING PRACTICE.[1]—IX.

IRONWORK.

Eagle des. for Napoleon I.—by C. Normand—after Raguenet.

IN this chapter I propose to enter into the manner of construction of the ironwork required in a fireproof building, also the assumed loads on the several floors and roof, and the general methods used in calculation of strength of same.

The first-floor framing is usually formed of iron girders, running between piers in basement built for this purpose and also as a bearing for first-story columns, and between or on these girders the iron floor-beams are laid.

Many of the interior walls of the building do not descend below the second-floor level, as most of the space on the first floor is required by the post-office department, and only such interior walls as are required to sufficiently stiffen the lower part of the building have their origin in the basement. In the second floor are the principal girders which rest on cast-iron columns, and on these girders are built the interior walls; these girders also support the second-floor beams.

In the second story are generally placed the court-rooms, and as they are always much larger than the office-rooms, it is customary to make them two stories in height, partly for architectural proportion and also for acoustic effect. As the court-rooms would be too wide to span with iron beams and oftentimes even with iron girders, the ceilings are supported by means of iron-trussed partitions, which divide the space over the court-rooms into office-rooms, but when the floor over the court-room is the attic floor, it is a common practice to suspend it from the roof-trusses.

The floors above the second story are carried by means of iron beams built into walls at their respective levels, except in the case of court-room ceilings and floors over them, which may be suspended by either of the methods before mentioned.

In many large public buildings there is a glass ceiling in the central part of building of the second floor level to light the post-office working-room; above this glass ceiling a well extends to the top of building, which is covered with a glazed roof, which lights the interior of the building and the colonnade at each floor.

The main roof is constructed of I-beam rafters and purlins; the roof framing is supported by trusses where required, and on the rafters are riveted the ⊥ iron purlins to support the 2″ terra-cotta tiles.

Dormers are usually constructed with angle-iron frames and cased with galvanized iron of suitable pattern to agree with the building.

The methods of construction and calculating do not vary materially from those in private practice, and only a short, general synopsis will here be given. The loads assumed for fireproof floors, including the weight of construction, are as follows:

First floor, . .	200 pounds per square foot.
Second floor, . .	180 " " " "
All other floors, . .	160 " " " " "
For the roof (weight of construction only) measured on slope, . .	50 pounds per square foot.

The wind is calculated with the normal pressure against roof surface as given by accompanying table of normal pressures for different angles with the horizon, on a basis of fifty pounds pressure per square foot against a plane at right angles to its direction.

Angle.	Normal Pressure per sq. ft.	Angle.	Normal Pressure per sq. ft.
5°	$6\frac{5}{10}$ lbs.	45°	$45\frac{1}{10}$ lbs.
10	$12\frac{1}{10}$ "	50	$47\frac{6}{10}$ "
15	$17\frac{5}{10}$ "	55	$49\frac{5}{10}$ "
20	$22\frac{9}{10}$ "	60	50 "
25	$28\frac{1}{10}$ "	65	$50\frac{7}{10}$ "
30	$33\frac{1}{10}$ "	70	$51\frac{5}{10}$ "
35	$37\frac{6}{10}$ "	80	$50\frac{5}{10}$ "
40	$41\frac{7}{10}$ "	90	50 "

Deck roofs and flat slopes are not calculated to resist wind-pressure, but are calculated for snow—from five pounds to twenty pounds per square foot of roof according to locality of building.

For floor-beams uniformly loaded, the tables in "*Carnegie's Pocket Companion*" are used, taking a maximum fibre strain of 12,000 pounds per square inch (*i. e.* factor of 4), and deflections not exceeding $\frac{1}{360}$″ per lineal foot of span to prevent cracking of plaster finish of ceilings underneath.

For unequal loading the girder or beam is calculated and the nearest size stronger is used, in case there is not one of exact strength.

As an example, we will take a room where the weight is equally distributed at 160 pounds to the square foot. The size of the room is 20′ x 30′, see Figure No. 37. It will be found more economical to span this room by a girder across the centre of the short side and frame the floor-beams into it than to use single beams spanning the shorter distance.

The beams are 5′ 0″ apart and 15′ 0″ span, so that the load to be carried by each single beam = 5′ x 15′ x 160 pounds per square foot = 12,000 pounds.

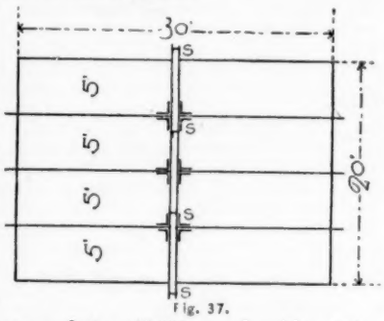


Fig. 37.

By reference to tables we find that a 9″ I-beam weighing 70 pounds per yard will carry 11,580 pounds equally distributed, a little less than the load, so taking the next size stronger, we use a 10″ I-beam 99 pounds per yard, which will carry 16,000 pounds equally distributed. We now take the girder: the load on it = 20′ x 15′ x 160 pounds per square foot = 48,000 pounds with a 20′ span. We find from the tables that one 15″ I-beam weighing 150 pounds per yard will carry 28,240 pounds; using two such beams side by side, the girder is equal to 28,240 pounds x 2 = 57,480 pounds, which is in excess of the load.

The load on the girder is assumed to be evenly distributed, because by calculation it is proved that it is quite safe to use tables of uniformly-loaded beams when the loads are equal and are transmitted at points equidistant on the girder, though several feet apart. The two beams composing the girder are kept the proper distance apart by cast-iron separators (marked *s* in Figure No. 37) bolted through flanges of beams, one separator at each bearing where the shearing force on webs is greatest and intermediate separators spaced not over 6′ 0″ apart.

We will now take the connections for this framing, allowing 7,500 pounds per square inch shearing strain on rivets, and 15,000 pounds per lineal inch bearing value on a pin or rivet one inch in diameter, which may be found also in "*Carnegie's Pocket Companion.*" The load on each beam = 12,000 pounds; one-half (6,000) is transmitted to the wall and the other half is carried to the girder. By table we find one rivet $\frac{5}{8}$″ diameter is safe for 2,300 pounds single shear, therefore

$$\left.\begin{array}{l}\text{Shear on one end of beam}\\ \text{Shearing value of one rivet.}\end{array}\right\} = \left\{\frac{6000 \text{ lbs.}}{2300 \text{ lbs.}} = 2\frac{6}{10}\right. \text{ rivets or 3 rivets required.}$$

The web of 10″ I-beam is $\frac{32}{100}$″ thick and bearing value of web on one $\frac{5}{8}$″ diameter rivet = 3,000 pounds, therefore

$$\left.\begin{array}{l}\text{Load on one end of beam}\\ \text{Bearing value of web on 1 rivet.}\end{array}\right\} = \left\{\begin{array}{l}6000 \text{ lbs.}\\ 3000 \text{ lbs.}\end{array}\right\} = 2 \text{ rivets required.}$$

By using a standard connection, as shown in foregoing table, we get bearing of $\frac{32}{100}$″ web on three $\frac{5}{8}$″ diameter rivets at 3,000 pounds each = 9,000 pounds and double shear on three $\frac{5}{8}$″ rivets = single shear on six rivets, which is in excess of what is required. The angle connections being each $\frac{3}{8}$″ thick = $\frac{3}{4}$″ for both, which gives an excess of bearing surface.

For bearings of beams on walls, the usual practice is to distribute the loads so that the pressure on brickwork shall be about 18,000 pounds per square foot. At all beam ends that transmit not over 12,000 pounds to their bearings is used a plate 8″ x 12″, the beam bearing the full width of 8″ on this plate. At the end of girder in the foregoing example, there are 24,000 pounds bearing on each end in wall, which will require a plate 16″ x 12″. No girder is given less than 12″ length of bearing. No cast-iron bearing-plate under a girder is less than 2″ thick and often greater. For example, there is a box-girder, one end of which transmits 144,000 pounds to a wall 2′ wide, allowing a pressure on brickwork of 18,000 pounds per square foot, a bearing-plate is required having an area = $\frac{144,000}{18000}$ pounds = 8 square feet = 2′ wide x 4′ long. As the girder is only 18″ wide, the bearing-plate extends 15″ on each side of girder, and unless made sufficiently strong to distribute the load, it will break. Treating the plate as an inverted beam fixed at one end (*i. e.*, in centre, with a projection of 15″ on each side), and uniformly loaded, we find weight on overhanging portion = 15″ x 24″ x 18,000 pounds per square foot = 45,000 pounds, and 45,000 pounds x $7\frac{1}{2}$″, centre of gravity or lever-arm of bending moment = 337,500″ pounds bending moment.

Formula:

$$\text{Depth}^2 = \left\{\frac{\text{Bending Moment}}{\frac{\text{Modulus of Rupture}}{6}} \times \text{breadth}\right.$$

[1] Continued from page 9, No. 628.

For an iron plate using a factor of 8, we have

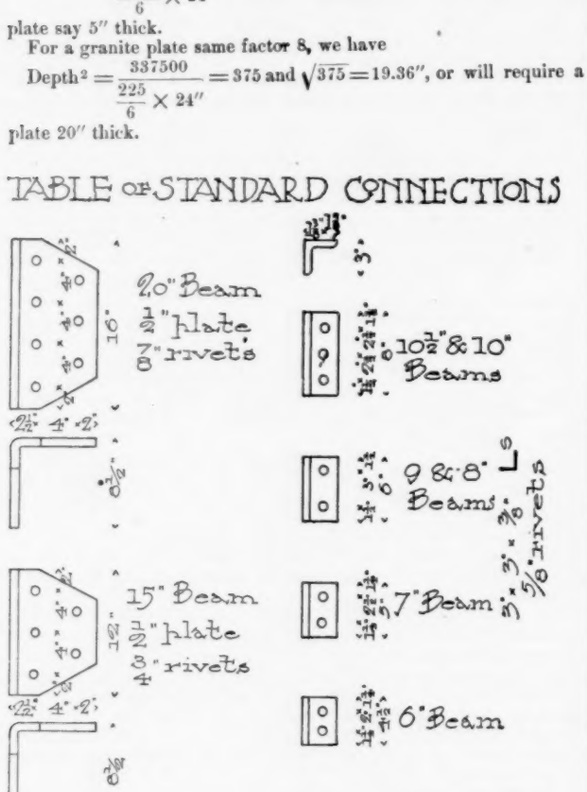
$$\text{Depth}^2 = \frac{337500}{\frac{4000}{6} \times 24''} = 21 \text{ and } \sqrt{21} = 4.5826'', \text{ or will require a}$$

plate say 5″ thick.

For a granite plate same factor 8, we have

$$\text{Depth}^2 = \frac{337500}{\frac{225}{6} \times 24''} = 375 \text{ and } \sqrt{375} = 19.36'', \text{ or will require a}$$

plate 20″ thick.



The modulus of rupture for other materials may be found in Trautwine.

In long spans and heavy loads it frequently happens that rolled beams have not enough depth for proper stiffness and are not as economical as plate or box girders built of plates and angles riveted.

Formulas used for plate-girders in the more common cases.

l = span.
x = distance from support to point where strain is required.
d = depth of girder between centres of gravity of sections of flanges.
w = uniformly-distributed stationary load per unit of length.
W = concentrated load at any point.
$S.\ fl$ = strain on either flange.
$S.\ wb$ = strain on web.
l, x and d must be all in the same terms.
The weight of the girder itself must never be omitted from the calculations of the strains.

Girders fixed at one end only.

Case 1. Load (W) at extremity.

$$\text{At the support } S.\ fl = \frac{Wl}{d}$$

$$\text{At the support } S.\ wb = W$$

$$\text{At any other point } S.\ fl = \frac{W}{d} \times (l - x)$$

$$\text{At any point } S.\ wb = W.$$

Case 2. Load (w) uniformly distributed.

$$\text{At the support } S.\ fl = \frac{w \times l^2}{2d}$$

$$\text{At the support } S.\ wb = w \times l$$

$$\text{At any other point } S.\ fl = \frac{w}{2d} \times (l \times x)^2$$

$$\text{At any other point } S.\ wb = w \times (l - x)$$

Girders supported (not fixed) at both ends.

No strain in flanges at points of support.

Case 1. Load (W) at centre.

$$\text{At the centre } S.\ fl = \frac{Wl}{4d}$$

$$\text{At any other point } S.\ fl = \frac{Wx}{2d}$$

$$\text{At any other point } S.\ wb = \frac{W}{2}$$

Case 2. Load (W) at any point, dividing girder in two segments, a = the shorter and b the longer.

$$\text{At the weight } S.\ fl = \frac{Wab}{ld}$$

$$\text{In the shorter segment } S.\ fl = \frac{W\ x\ b}{ld}$$

$$\text{In the shorter segment } S.\ wb = W\frac{b}{l}$$

$$\text{In the longer segment } S.\ fl = \frac{Wxa}{ld}$$

$$\text{In the longer segment } S.\ wb = W\frac{a}{l}$$

In the latter case x must always be measured from that support which is on the same side of W as the point where the strain is required.

The sections with single webs are more economical than those with double webs (box-girders), but the latter are stiffer laterally and should always be used where a great length of span requires a wide top flange.

The web of the girder must be made of such thickness that there will be no tendency to buckle, and that the vertical shearing stress per square inch will not exceed 9,000 pounds. Security against buckling is attained when this shearing stress does not exceed $\frac{10000}{1 + \frac{d^2}{3000\,t^2}}$, in which d represents the depth of web, and t its thickness in inches. The web should also be stiffened at intervals by vertical angle-irons, and at bearing ends of girder, webs should have reinforcing plates between top and bottom angle-irons.

The rivets should be ⅞″ unless the girder is light, when ¾″ diameter rivets may be used; they should not be spaced more than 6″ apart and should be closer for heavy flanges, but in all cases they should not be spaced farther than 3″ or 4″ at ends for a length not less than the depth of the girder.

A strain of 10,000 pounds per square inch should be allowed both for tension and compression on the gross area of flanges.

All cast-iron columns are made with a factor-of-safety of 8.

The general formula for strength is

$$W = \frac{10000}{1 + \frac{1}{400}\left(\frac{l}{d}\right)^2} \times \text{area in inches}$$

l = length of column in inches.
d = least diameter in inches.
w = safe load in tons.

Complete tables of columns and their loads may be found in the "*Pocket Companion*" of the Dearborn Foundry Company.

Roof trusses are designed to suit shape of roof, and are usually very irregular both in shape and loading.

$$\text{The strain allowed in compression members for square ends is } \frac{8000}{1 + \frac{l^2}{40000\,R^2}}$$

l = length of member in inches.
R = radius of gyration in inches.

For pin and square bearings substitute 30000 for 40000 and for both pin bearings use 20000.

The strains allowed in tension members are for dead loads:

General bracings, 12000 pounds per square inch.
Wind " 14000 " " " "
Suspenders from 8000 to 10000 pounds per square inch.

Rivets are taken at 7500 pounds shearing strain and at 15000 pounds bearing value.

Pins are taken at 9000 pounds shearing strain, 14000 pounds bearing value, and 18000 pounds bending. J. E. Blackwell.

[To be continued.]

THE UNDERPINNING OF THE GREAT YARMOUTH TOWN HALL.

IN 1882 a new block of municipal offices and law courts was built at Great Yarmouth, from designs by Mr. J. Bond Pearce, of Norwich. The new structure, which was illustrated in *Building News* on the 27th September, 1878, measured 132 feet by 108 feet by 50 feet high, to the parapet, with a clock-tower 110 feet high, the whole weighing about 5,000 tons, costing £30,000. Its architecture was modern Queen Anne, and its construction was red brick above string-course, and red St. Bees stone facing at base. The subsoil consisted of a gravel bank, on which was a thickness of 16 feet to 18 feet of ooze, surmounted by about five feet of made ground. The river Yare was 70 feet distant. The structure gave early indications of unequal subsidence, and in 1886 the movement apparently approached the limit

of safety, and steps were taken to underpin that portion which had settled most by the insertion of concrete blocks beneath the foundation; but the attempt was abandoned, as also was a proposal to widen the foundations by the insertion of wrought-iron needles through the brickwork, and supporting them near the ground on concrete blocks. By November last the west front of the building had sunk over a foot at its ends and eight inches at its centre, and there were some ugly fissures in it, as well as in the north wall, and the walls and tower were leaning towards the river. The Town Council, therefore, resolved to demolish the western portion of the block with the view of its being re-erected on more stable foundations. The cost of pulling down and rebuilding would have exceeded double the sum hitherto expended, and the estimated time for operations was three years.

A scheme was then submitted to the Town Council by Mr. Duckham, of the Millwall Docks, Engineer, in conjunction with Mr. James E. Teasdel, Engineer and Surveyor, of Great Yarmouth, for preventing the further subsidence of the crippled portion of the block, and lifting the portions which had settled most to the level of that which had settled least. The proposal also included the straightening of the bulged walls. The prominent features of this scheme were: First, cast-iron screw cylinders (those as adopted varied from two feet six inches to three feet diameter) placed at intervals of about nine feet inside and outside the main walls, and screwed down into the ballast, then filled with cement concrete. Secondly, double lines of wrought-iron girders I I on top of the cylinders parallel with the walls. Thirdly, wrought-iron needles 14 feet to 16 feet long, mostly 16 inches by 6 inches I with a top table 12 inches wide, and averaging eight feet long, equal to width of the concrete under brick footings, passed through under this concrete foundation at intervals of about three feet six inches, and suspended at each end by two two-inch bolts from the before-mentioned I I girders. The insertion of these needles and the tightening of the bolts, transferred the weight of the building from the unstable ground on to the screw-piles. The tower weighing 700 tons and having a base 20 feet square, was to be somewhat differently treated. Five special cylinders of four feet six inches and five feet diameter were proposed for this to be placed as the adjacent walls permitted. The eastern and western walls of the tower — *i. e.*, those without openings on the ground floor — to be each sandwiched between a pair of massive lattice girders below the floor-level. Needles of I I section to be inserted through the walls and through the girders, and bolted up to the upper member by four two-inch bolts at each end. This proposal of Messrs. Duckham and Teasdel was adopted, after severe criticism and opposition, and a contract for the chief portion of the work was entered into by the Town Council with Mr. Thos. Gibson, of Westminster Chambers. The first cylinder was pitched at the end of May last, and the work was completed in November. The whole western portion of the block and tower are supported on a gridiron of wrought-iron joists, suspended from the girders which rest on the pile-tops. The suspending bolts have nuts at each end and 12 inches of thread; by a systematic and gradual screwing up of some more than others, the low parts of the block have been lifted to the higher level, the unsightly curves have been taken out of the building, and the tower set upright by raising its lower side. The ground has been cleared to a depth of two feet under the old concrete, and the trenches have been filled in with cement concrete, forming one mass, encasing the pile-tops, girders and bolts, thus forming a new foundation seven feet wider than the former one, irrespective of the support given by the immovable screw-piles. The tower-girders have been similarly concreted. The replacing of the floors, making good the damaged masonry and refitting in general are now being proceeded with. The damaged places in walls are being cut out through the entire thickness, the wall and windows adjoining the tower being rebuilt all in Portland cement. The subsidence in walls of main hall and tower during these operations was only nominal, and was rectified by the lifting process. What settlement did occur during the progress of the works was chiefly due to the flooding of the trenches in July last, and was at once stopped by the insertion of the cross-girder needles. The building will be ready for re-occupation by March next, the time thus occupied having been ten months, against the contemplated three years. The total cost of the work and its contingencies will be well within the estimate, £8,250. — *Building News.*

FIRTH OF FORTH BRIDGE.

THE construction of the Forth Bridge has reached a stage at which it is possible to estimate with some degree of accuracy when this gigantic engineering work will be completed; and the time given by the resident engineer is towards the end of next year. Describing the progress of the work, the *Scotsman* says that the huge "straddle-legged" structures which are reared upon the three large piers were practically completed before the end of the year. These rise to the great height of 360 feet above high-water mark, and form the *points d'appui* or structural bases from which the "cantilevers" are being built out on either side. The term cantilevers is applied to the wing-shaped structures which shoot out from the large pier-towers and reach towards one another across the great spaces that have to be spanned. The building of these cantilevers has at present reached a most interesting stage. From the summit of the towering-pier structures, arms are being stretched out on each side into mid-air. These are each composed of two pairs of huge booms, constructed on the girder or lattice-work principle, and project sheer above the waters, which are full 300 feet below. There is not the slightest under-support; the whole fabric relies upon the strength of the ties which bind it to the top of the cantilever towers. Incredible as it may appear, these top-members have been built out to the distance of 125 feet, and seem just now to hang in a precarious position like the ash upon a half-smoked cigar. It is in this part of their work that the designers and contractors claim that they have vindicated the soundness of their calculations. It was in the springing of these top-members or arms, and carrying them out unsupported the necessary distance, that cavillers and detractors of the scheme have chiefly propheized failure. Yet from both the Queensferry and Fife pier-structures these aerial platforms have been built out, two from each, without a hitch, to the distance of 125 feet. They now await the rearing of a temporary column which is running up from below, and which will form a support, enabling them to be carried still further out till they meet the first permanent supports — the cross struts or tubes which spring from the bases of the pier-towers. The lower arms, meantime, have been proceeding even more rapidly. These are the immense tubes which protrude outward and upward, and along which the pressure exerted by the weight of the spans is carried back to the stone piers. They have been carried out to the distance of fully 160 feet. It may be explained here that the top-arms, which are constructed on the girder system, are the tension members — that is to say, are subjected to a pull by the weight of the structure which they support, while the lower arms, which are of tubular construction, are the compression members, and bear the down push, as it were. These lower tubes have been built out in exactly the same way as the upper ones — *i. e.*, without any support from underneath. The workers to-day are practically standing upon their labors of yesterday. As soon as a fresh round of steel plates is added to the tubes, or an additional girder section riveted to the top-arms, the platforms, with their freight of men and cranes and other mechanical appliances, are slid out correspondingly and a new piece of work is begun, which again when completed will give the necessary standing support for a further extension. Indeed, this is characteristic of the whole of the work at the bridge. Every piece of work done becomes the basis of another advance.

The point, however, of this great work of engineering which is of preëminent interest, and about which speculation and prophecy are most busied, is the problem of bridging over the two great spans of 1,710 feet each, which extend north and south of Inch Garvie. The largest Tay Bridge spans are just 245 feet, so that the proposed spans at Queensferry will be seven times as long. The largest span of any bridge in this country is that of the Britannia Bridge, over the Menai Strait, which is 460 feet in length, or somewhat over a quarter of the size of the Forth Bridge spans. To throw a single span across such a space as 1,710 feet would, even under ordinary conditions, be a tremendous undertaking. But in the present case the difficulties are incalculably enhanced, because the whole distance has to be bridged over without any support from below. The water is 200 feet deep on each side of Inch Garvie, so that it is impossible to found temporary scaffolding or piers upon the sea-bottom, nor can any device of pontoons be of practicable service. The problem, then, comes to be simply this — to join the Inch Garvie pier with the north and south piers (each 1,710 feet apart) by simply building straight out across the waters, at a height sufficient to allow the largest ships to pass freely up and down the Firth. It is this that explains why the immense "straddle-legged" towers which are the prominent features of the structure just now, have been reared so high as 360 feet. These great spans will never be entire — that is, the two halves of each will never be joined. Each half of each great span will hang entirely by its own supports on the main piers. Owing to the large expansion and contraction of so immense a structure of steel under changes of temperature, it would endanger the fabric were it actually joined. At a certain point in the central viaduct (as the 150-feet high girder structure along which the train passes is called) the plates will overlap each other, so that the shrinkage caused by the cold may not make a gap, nor the expansion in the hot weather cause "buckling." The extreme variation in the length of the 1,710 feet spans under alterations of temperature may not exceed nine inches, but provision is made for 18 inches. The weight of one of the spans will be about 16,000 tons. The heaviest possible addition from trains — if we take the extreme case of two coal-trains standing side by side in the middle of the span, and weighing 400 tons — would be only five per cent of the dead weight. In this way it is estimated that under a passing train the bridge will stand as stiff and firm as if the train were not there. Wind is a much more serious calculation than the trains, as with the maximum-pressure (56 pounds per square foot) the large spans be subject to an additional side-pressure of 2,000 tons. The widest margin, however, has been left in all cases.

Hydraulic Elevators in London Hotels. — One of the colossal hotels in London is the Metropole. An idea of its size is given by the fact of its having seventeen elevators. These "lifts" are worked by water from the mains of one of the hydraulic-power companies, of which a number are established in England. By them water under pressure is carried beneath the streets and delivered to consumers at fixed rates After doing the work required of it, the water is returned to the central station of the companies, there to be used over again. — *Exchange.*

SOCIETIES

EXTRACTS FROM THE MINUTES OF MEETING OF THE CHICAGO CHAPTER, AMERICAN INSTITUTE OF ARCHITECTS, March 22, 1888.

Dinner served at 6 P. M., at conclusion of which, business meeting. Three applications for membership were favorably considered.

A printed extract from the minutes of Feb. 2d of the New York Chapter, relative to the proposed competition for the Criminal Courts' Building in that city, was read. By a unanimous vote, the Secretary was instructed to express to the New York Chapter, the hearty concurrence of Chicago Chapter in the opinions expressed in those minutes.

A communication from the Board of Trustees of A. I. A. relative to the claims of the late Thomas U. Walter against the U. S. Government was referred to a Committee.

James R. Willett, Architect, read a paper on "Graphical Statics applied to Architectural Questions." At the conclusion of the paper, a vote of thanks was moved and Mr. Willett was begged to furnish copies of his paper for publication. Carried unanimously.

Adjourned.

W. A. Otis, *Secretary.*

NOTES AND CLIPPINGS

Changes near the Sorbonne, Paris.—The builders, or rather the demolishers (the Paris correspondent of the London *Daily Telegraph* says), are now clearing away in order to make room for the further extensions to the new wing of the Sorbonne, another series of venerable slums. Among the first places to be improved off the face of the locality are the Rue Gerson and the greater part of the Rue Saint-Jacques. The Rue Gerson is famous as being that in which Pascal penned his Provinciles almost under the windows of the stronghold of the Jesuits. It was there that attempts were made to intimidate Pascal and to evict him from his humble tenement after his writings and books had been burned by the public executioner. In the Rue Saint-Jacques was once the famous tavern of the Cochon Fidèle, which was a meeting-place and a museum of Bohemia. There, too, were the hotel, or rather the lodging-house, which sheltered Rousseau, and the old book-shop of Mère Mansut, whose establishment was crammed with literary wares and had neither doors nor windows. The proprietress used to sleep on a pile of books and perform her ablutions in the street. When a customer asked for a volume she used to go straight to the place where it lay amid a colossal collection of other tomes. Students were sometimes in the habit of making bets as to whether or not Mère Mansut would be able to find within a given time some old volume of forgotten lore which reposed amid ancient dust in the dark recesses of her shop.

Fathomless Quicksands.—In the construction of the Kansas Pacific and Atchison, Topeka and Santa Fe Railroads, says the Wichita (Kan.) *Beacon*, one difficulty of frequent occurrence which was met with was the quicksands. From western Kansas to the mountains, quicksands are to be found in nearly every stream, no matter how small, and to successfully bridge them required an expenditure out of all proportion to the size of the stream to be crossed. Pile-driving was tried, but the longest piles disappeared without touching bottom. Then filling with earth and stone was attempted and met with equally poor success, as the quicksands were apparently capable of swallowing the entire Rocky Mountains. The only means of crossing a quicksand was found to be to build short truss bridges across them. This was very expensive, but was the only thing to be done. As instance of the practically bottomless nature of the quicksands, the case of an engine that ran off the track at River Bend, about ninety miles from Denver, on the Kansas Pacific may be cited. The engine, a large freight, fell into a quicksand and in twenty minutes had entirely disappeared. Within two days the company sent out a gang of men and a wrecking train to raise the engine. To their surprise they could not find a trace of it. Careful search was made, magnetized rods were sunk to the depth of sixty-five feet, but no engine could be found. It had sunk beyond human ken, and from that day to this has never been discovered. Cattle and horses are frequently lost, the only animal that is safe being a mule. No greater instance of the intelligence of this much-maligned quadruped can be cited than the skill and care with which it avoids all unsound bottom. As its hoofs are much smaller and narrower than those of a horse, it would mire down in places where a horse could safely pass. Recognizing this fact, whenever a mule feels the ground giving way under its feet it draws back instantly and cannot be induced to advance a step, although a whole drove of horses may have immediately preceded.

TRADE SURVEYS

The conservative course which railroad builders, managers, investors, manufacturing interests, and jobbing and retail interests throughout the country, have pursued during the past three months have been fruitful of some beneficial results, which perhaps are not apparent on the surface. Traders and manufacturers during the past thirty days have been inclined to complain of the downward tendency in prices and to regard this tendency as a drawback, and attribute it to tariff agitations, rather than speculation, over railroad-building, overbuying and too long credits. The fact of the matter is that the downward tendency is due entirely to different causes, although the influences named may have had some effect. The business public have seemed to act with one accord during the past winter restricting purchases, and, to some extent, in restricting manufacturing operations. For instance, in the iron trade, the output has declined some 20 ovens. The Pennsylvania production suffers more than the production of any other State. In the South, the output of furnaces, mills, foundries, factories, are increasing. Throughout the West a diversified demand for the production of mills and furnaces has preserved a more healthful trade condition, and the distribution throughout the winter has been somewhat larger, *per capita*, than in the East. In fact, Pennsylvania is beginning to suffer in the iron competition with the West and South, and only last week, a meeting of influential iron-makers was held for the purpose of begging favors from the railroads to enable them to hold their own against some new sources of supply. In the textile industry, the downward tendency has been restricted to products of wool, while in cotton goods the prices have been firm throughout the season, and even an upward tendency is claimed by a good many jobbers. In the boot and shoe industry prices have been steady for sometime past, but taking the trade one thing with another, the tendency in future production is downward. The same applies to the locomotive works, car-works, general machinery work, outside of valuable patents and processes. Wood-working machinery has declined slightly, owing to the considerable increase in mill capacity. The lumber manufacturing interests, especially in the northwest, anticipate a firmer price-list this season, but sales have been made within the past two weeks, at the larger Eastern distributing centres, that show their anticipations are doomed to disappointment. We find this tendency in nearly all industries. In a few there are exceptional influences at work to strengthen prices. All this is good for consumers. The upward tendency in price would naturally restrict enterprise, while the downward tendency broadens the foundation for increased expenditures. Within the past few days a number of things have begun to bud, to show that the downward tendency alluded to has about exhausted itself. The first inquiry of the business man and manufacturer is, will prices now advance. To this, it is safe to make a positive answer, that they will not. The reasons are: first, the depleted condition of orders with manufacturers generally will make them all anxious to book business for the summer and fall. Then, again, the producing capacity has been steadily enlarged since last autumn, besides competition is more active now, and producers of all kinds of products and va ue are naturally anxious to secure the season's business before their neighbors. The financial question bears both directly and indirectly upon the price tendency. The business interests are slow to observe the causes at work far below the surface. In the money markets while there has been considerable increase of currency, through bond-purchases, the actual volume of available *per capita* circulation, is less than it was two or three years ago, as measured by the business requirements. One evidence of this, is in the great increase in obligations made. Another evidence is shown in the increasing volume of money borrowed for Western farm, manufacturing and general business requirements. Another is the increasing volume of book-indebtedness between retailers, jobbers and wholesalers. There are a great many complaints of a scarcity of money in small trading circles, but no immediate harm can grow out of this, but the ultimate possibilities are that the declining supply of money will be felt seriously in trade, financial and manufacturing centres. This is a matter to which the law-making power of the country should give its careful attention, but to all appearances this vital question will remain for future consideration.

Building operations in a good many States have been rather sluggish. In Philadelphia the volume of new business is thirty per cent less than at this time last year. In Pittsburgh, thirty-five per cent, in St. Louis twenty to twenty-five per cent. The projected work in a number of cities west of the Mississippi River is even larger than last year, although there are some doubts as to whether all the work projected will be undertaken before May 1st. By that time an intelligent opinion can be expressed. One encouraging feature is that the brick-makers have sold all their stock, and have orders for more. Another is that manufacturers of building-hardware have made large contracts and are pretty well sold up. Nails have begun to move freely, and nail-makers speak well of the coming season. Contracts for shingles, laths, slate and a variety of products entering into building construction, show that in a general way building activity will not fall much behind last year if any, especially for the next three months. The workmen, except in a few cities in the West, are returning to work at old prices, and there will be no labor outbursts. Indications from Eastern and Western mining regions show that the production of coal will be fully as heavy as last year. Some of the older fields will produce less, but the spread of industries throughout the West will increase the demand in the new quarters. The mines of the South will increase their output by twenty-five per cent. Mines west of the Mississippi will be worked to their fullest extent. Makers of mining machinery are authority for the statement that mining operations will be developed on a large scale. The company producers are quarrelling among themselves, and the stronger faction is crowding the weaker to the wall for the purpose of compelling it to agree to an advance of $1.25 to $1.50 per ton at ovens. Within twelve months the coke-making capacity will be increased by two thousand ovens. There will be greater diversification of industries in the West this year than last, stimulated largely by the capital, which has been and will be borrowed for the purpose of planting new industries and enterprises along the line of the new roads. Railroad-building prospects at this time are far from favorable, yet there are a number of large enterprises projected within the past thirty days. It is needless to say that this will be a poor railroad-building year, though no one can speak with any certainty. There is an abundance of quotable stock for investment, and there is certainly much room for additional mileage, and the only safe opinion to entertain is that much building will be done. There are lots of small roads projected between the Atlantic and Mississippi to develop good traffic-producing territory in those States. Railroad-building is in its infancy yet, and the year 1888 will develop much more enterprise in that direction than is now apparent. The managers of other lines contemplate the construction of factories, and two or three roads are now entering upon that work. A great deal of railroad material will be contracted for within the next sixty days. Bridge-work will be prosecuted on an extensive scale. Within a few days the Pennsylvania structural iron-makers have received inquiries for several very large lots of material. The Standard Oil Company will build a two hundred mile pipe line, and if their projected operations are prosecuted, the mileage this year will reach between seven and eight hundred miles. Demand will come from unexpected sources. Those who were pretending to predict as to the volume and course of trade this season are astray simply because of the difficulty of forseeing all of the multitudinous requirements which a country like the United States is constantly creating.

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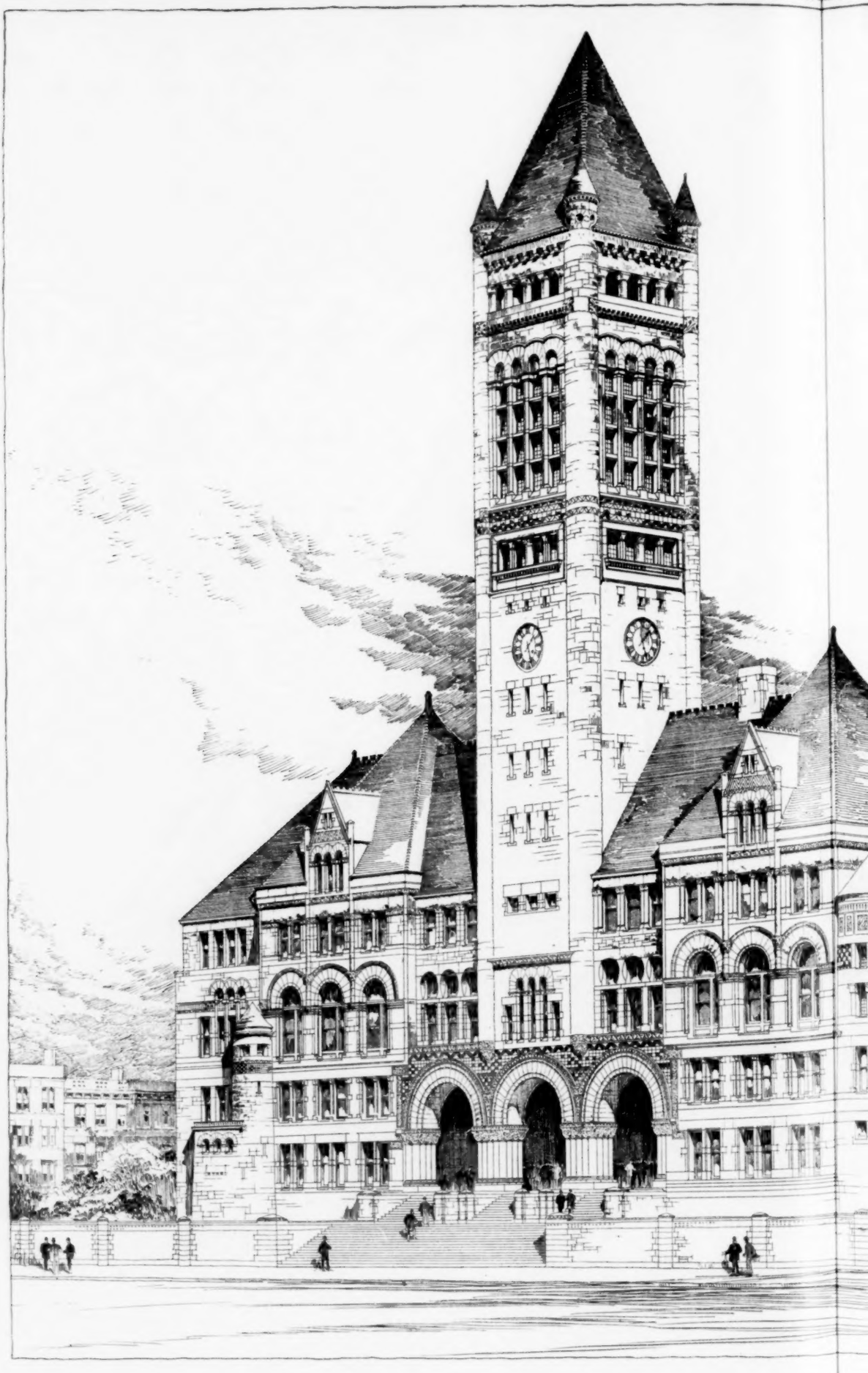

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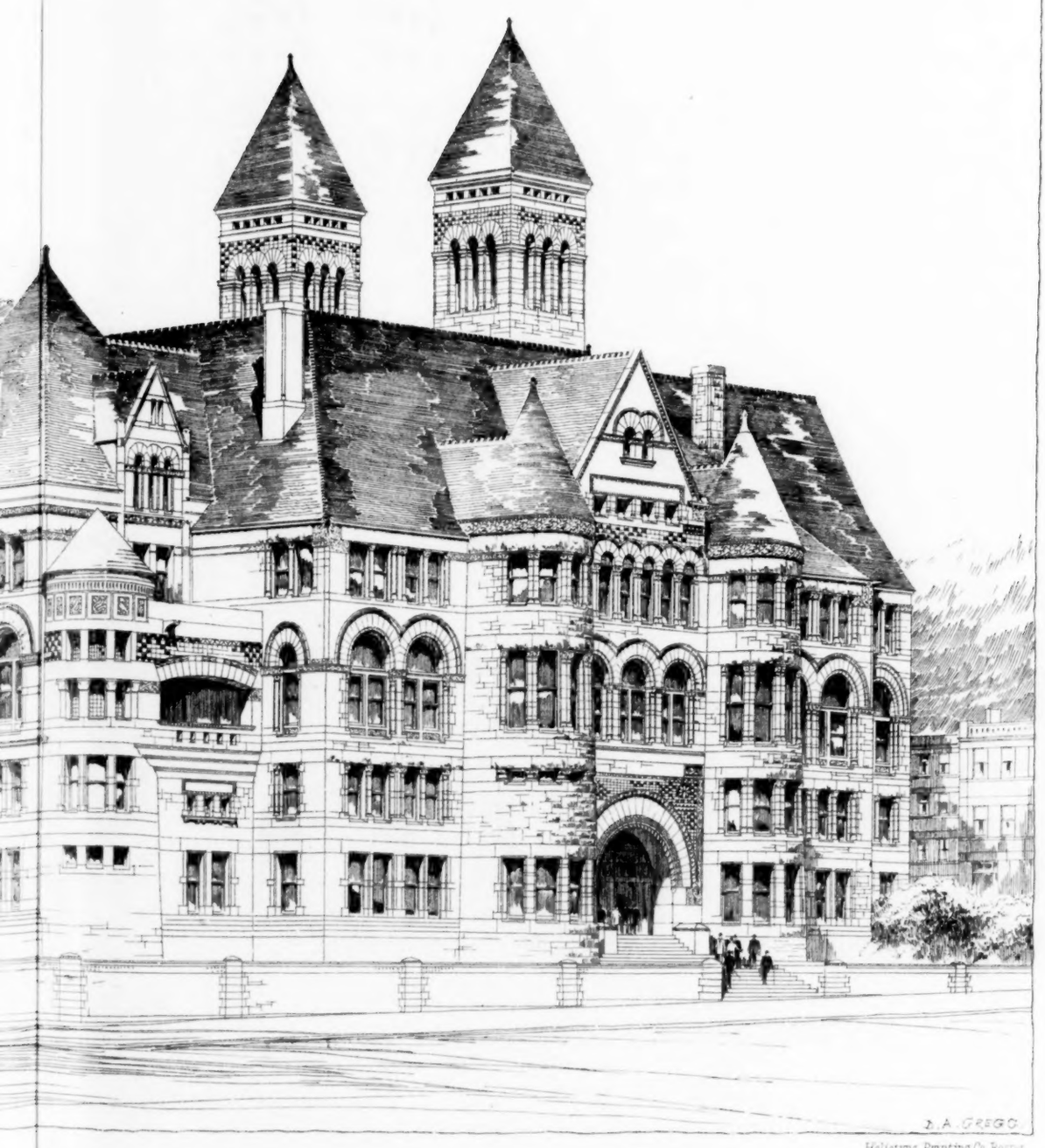

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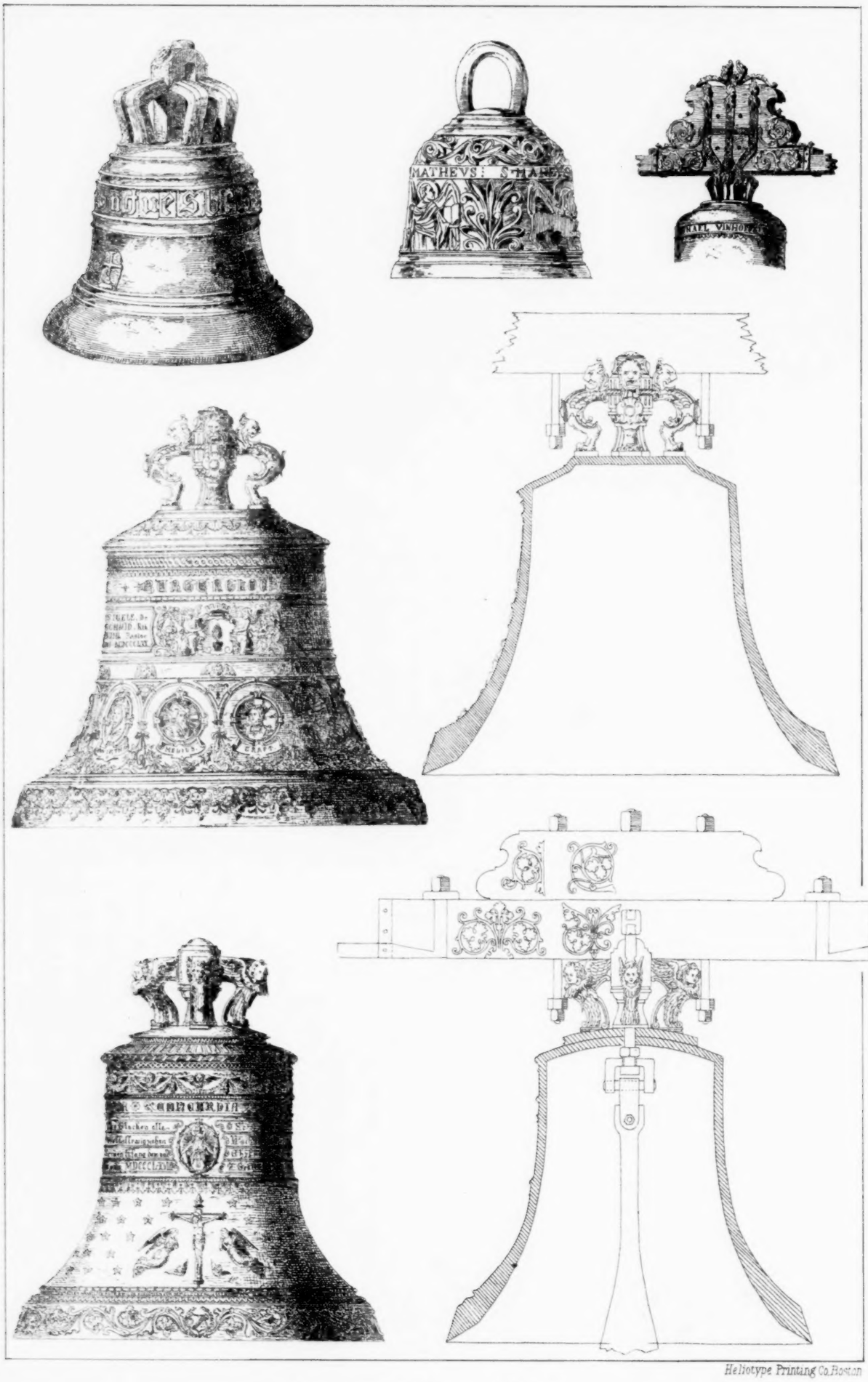

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Some Old Bells

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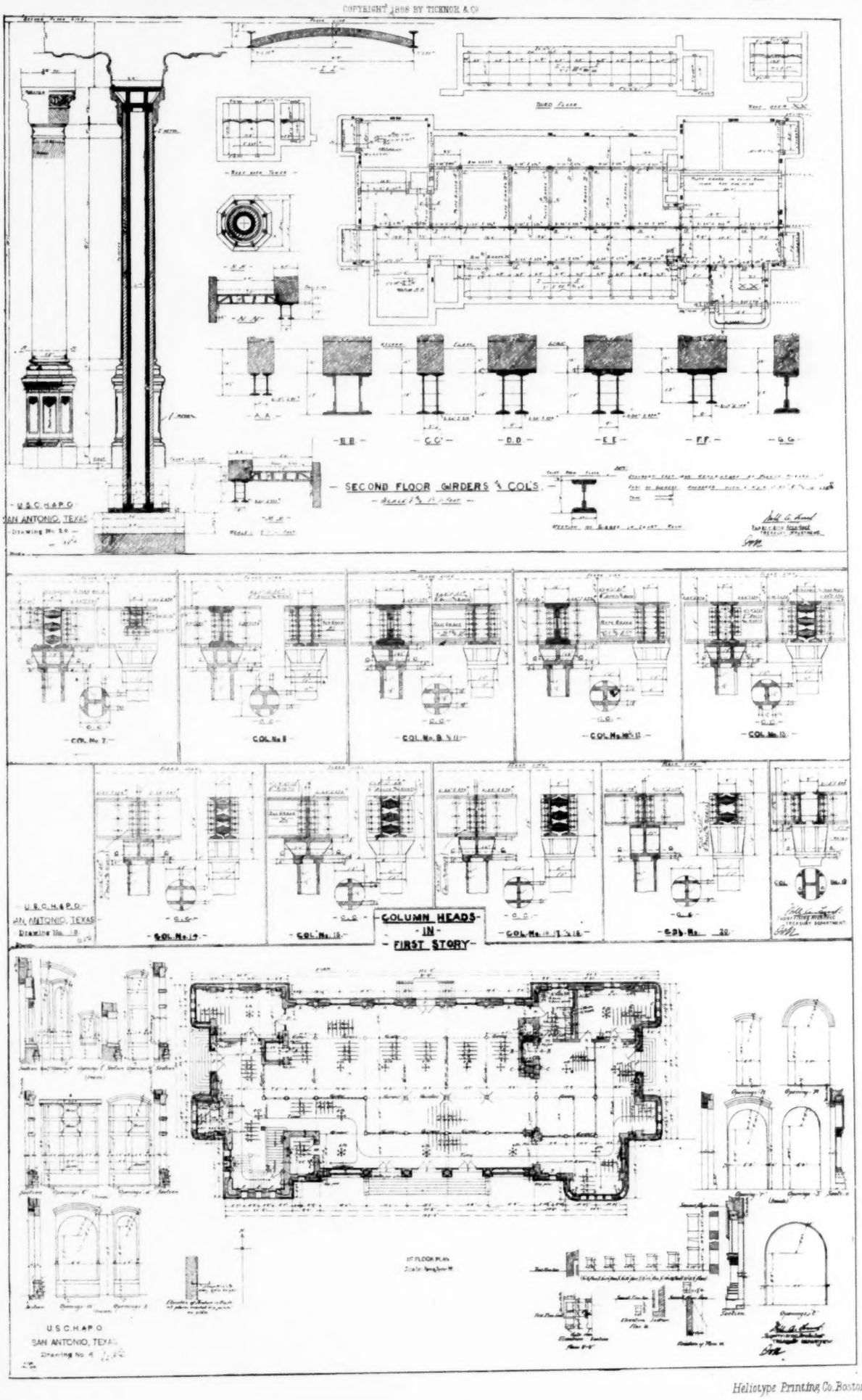

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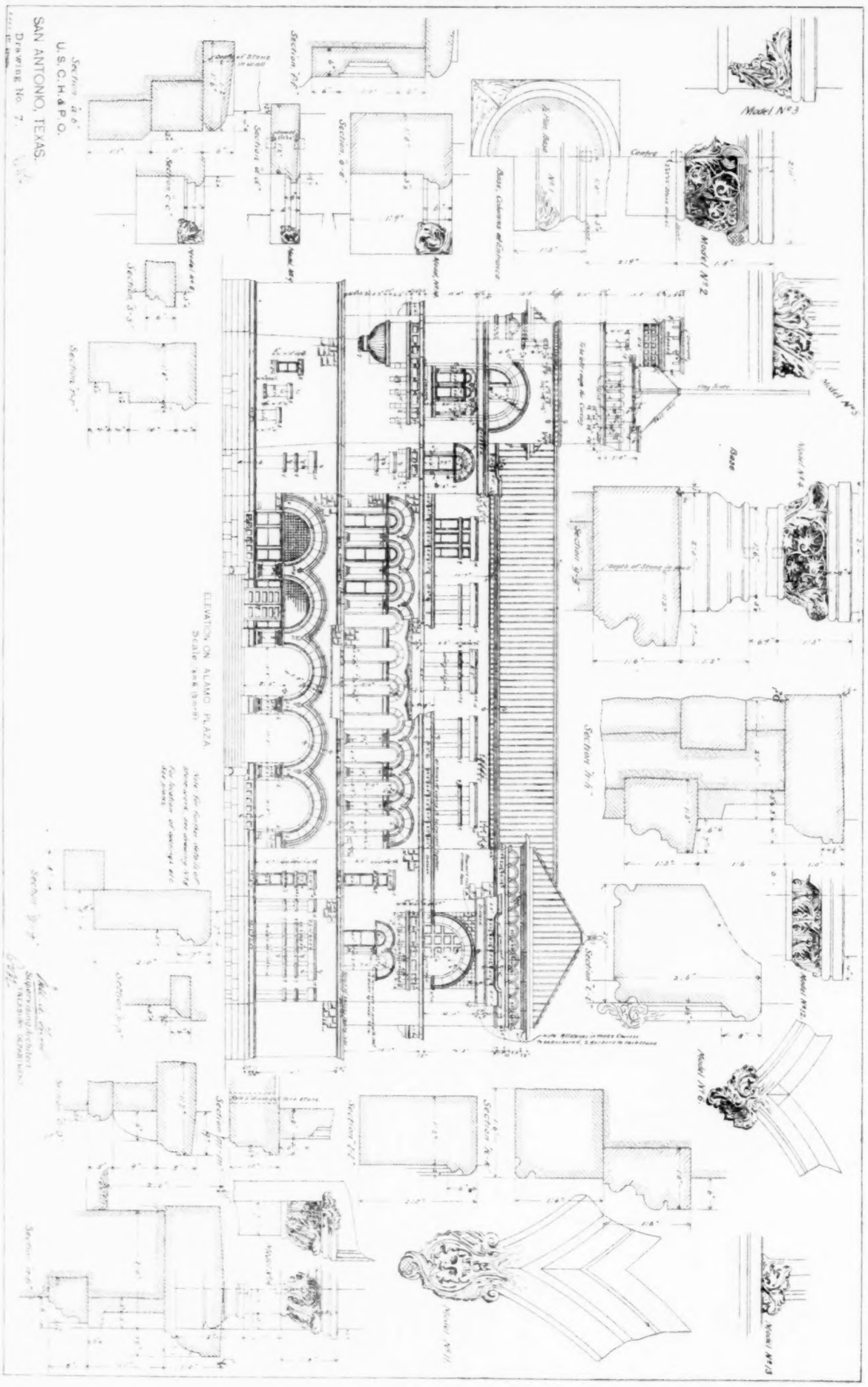
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Model No 4
Model No 5
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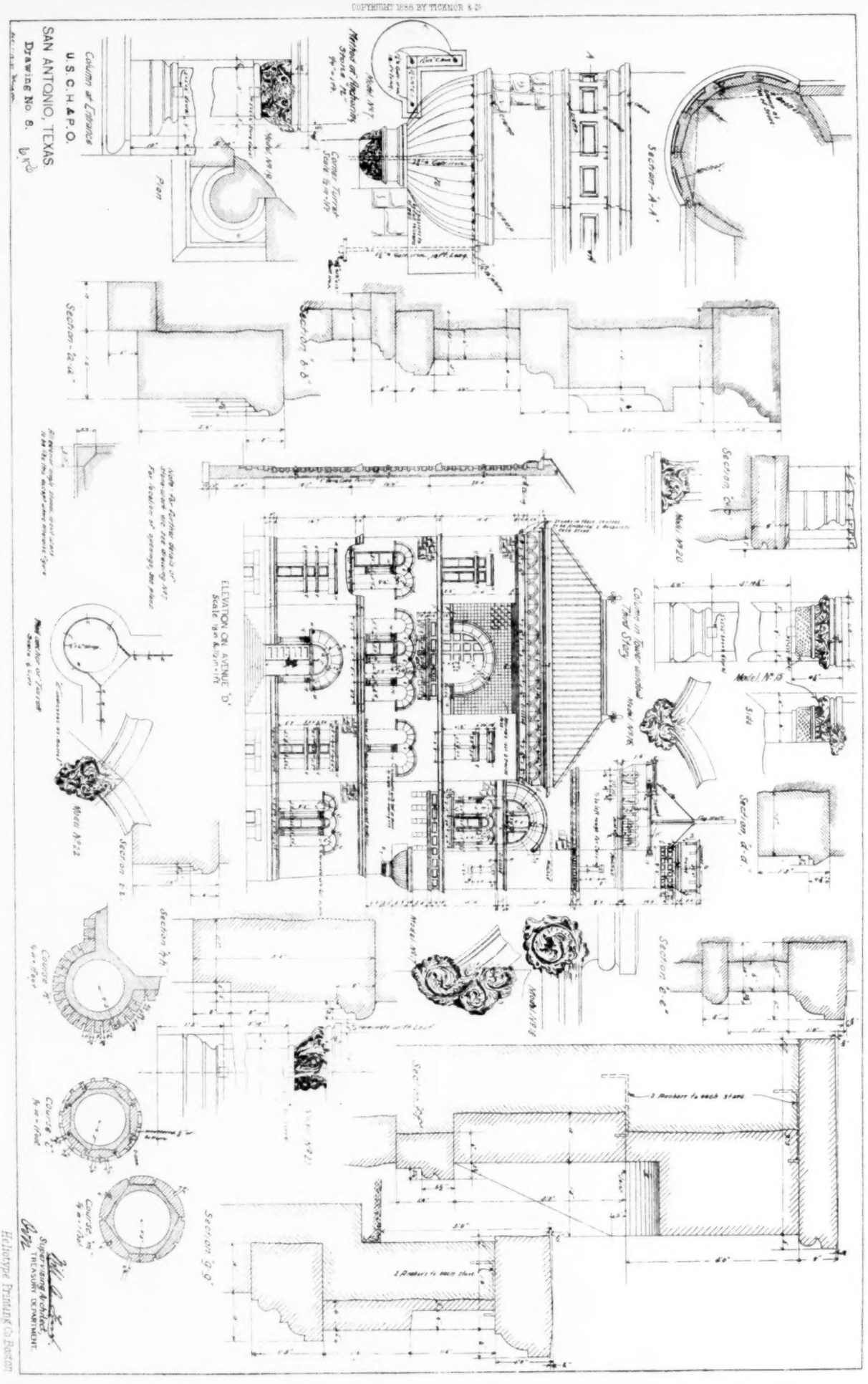
Centre

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